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A SERIES OF TEXTBOOKS FOR PERSONS ENGAGED IN PROFESSIONS AND TRADES OR FOR THOSE WHO DESIRE INFORMATION CONCERNING THEM. FULLY ILLUSTRATED

ELEMENTS OF STEEL REINFORCEMENT
SYSTEMS OF STEEL REINFORCEMENT
CONCRETE BEAM AND COLUMN DESIGN
REINFORCED-CONCRETE BUILDINGS
ENGINEERING STRUCTURES IN REINFORCED CONCRETE
OFFICE PRACTICE IN CONCRETE DESIGN
REINFORCED-CONCRETE SPREAD FOOTINGS
REINFORCED-CONCRETE CANTILEVER FOUNDATIONS
FORM WORK
SPECIFICATIONS AND COST OF REINFORCED CONCRETE
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PREFACE

The volumes of the International Library of Technology are made up of Instruction Papers, or Sections, comprising the various courses of instruction for students of the International Correspondence Schools. The original manuscripts are prepared by persons thoroughly qualified both technically and by experience to write with authority, and in many cases they are regularly employed elsewhere in practical work as experts. The manuscripts are then carefully edited to make them suitable for correspondence instruction. The Instruction Papers are written clearly and in the simplest language possible, so as to make them readily understood by all students. Necessary technical expressions are clearly explained when introduced.

The great majority of our students wish to prepare themselves for advancement in their vocations or to qualify for more congenial occupations. Usually they are employed and able to devote only a few hours a day to study. Therefore every effort must be made to give them practical and accurate information in clear and concise form and to make this information include all of the essentials but none of the non-essentials. To make the text clear, illustrations are used freely. These illustrations are especially made by our own Illustrating Department in order to adapt them fully to the requirements of the text.

In the table of contents that immediately follows are given the titles of the Sections included in this volume, and under each title are listed the main topics discussed. At the end of the volume will be found a complete index, so that any subject treated can be quickly found.

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ELEMENTS OF STEEL REINFORCEMENT

THEORY OF STEEL REINFORCEMENT

INTRODUCTION

1. With the cheapening of Portland cement and its extensive manufacture in the United States, reinforced concrete has entered largely into the construction of buildings and engineering structures. While the principles of reinforcing concrete with steel rods, bars, or other forms of sections remain the same, numerous methods of combining the steel and concrete have been devised.

In many instances, the different methods of construction have been based on peculiarities in the form of the reinforcing bars, it being considered by the advocates of the several systems that a deformed bar, or a bar provided with projections, corrugations, or imprints, furnished a greater hold in the concrete and, consequently, provided additional strength and durability. In each instance an effort was made to procure a deformation that would not reduce the strength of the bar and at the same time not materially increase its weight, for such an increased weight would prevent competition with the more economical forms of construction.

Other systems of construction depend for their distinguishing features on the method of placing the steel in the actual operation of construction. In these systems, the designers devoted their attention to the lacing, or tying

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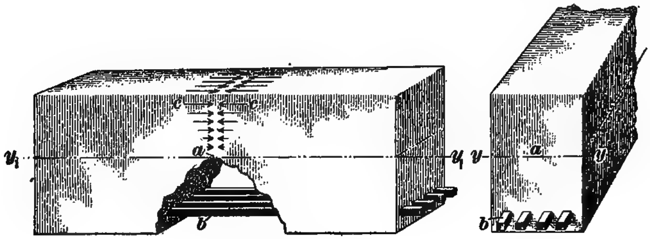
together, of the steel bars forming the reinforcement and devised methods of securing this reinforcement in the forms, or else they supported the forms from the steelwork without additional centering.

The several kinds of steel reinforcement in general use at present will be described in the ensuing articles, while the systems embodying the method of placing and assembling will be treated in *Systems of Steel Reinforcement.*

RELATION OF STEEL TO STRESSES PRODUCED

2. Principles of Construction.—When a beam is subjected to transverse stress, the portion of the beam section above the neutral axis is in compression, while in that portion below the neutral axis, tensile stresses are created. Ordinarily, concrete is about ten times as strong in compression as it is in tension. Thus, it can readily be seen that a beam of plain concrete without steel reinforcement would fail primarily from lack of tensile resistance, without realizing its full compressive strength. In order, therefore, to make concrete an economical material to use in construction, its deficiency in tensile resistance must be made up by embedding steel rods, bars, or some other form of metallic reinforcement in that portion of the beam section subjected to tensile stress.



(a) (b)

FIG. 1

In order to explain more fully this primary principle of reinforced concrete, reference is made to the reinforced, rectangular concrete beam shown in Fig. 1. The neutral line of the section is shown at $y_1\ y_1$ in the side view (*a*), while the neutral axis is represented by $y\ y$ in the end view (*b*). When the concrete beam is under transverse stress, there is neither tensile nor compressive stress at the neutral axis. Therefore, the point in the beam marked *a*, which is on the neutral axis, is subjected to zero stress.

Imagine that the concrete is cut away below the neutral plane, leaving the steel reinforcing rods, or bars, exposed as at *b*. It is evident that the strength of the beam is not much affected by the cutting away of the concrete in this manner, as the necessary tension below the neutral axis is supplied by the reinforcing rods of steel, while the necessary compression above it is furnished by the concrete, as at *c*, *c*. The amount of compression in each square inch of concrete above the neutral axis varies from zero at the axis to maximum at the extreme upper surface, or edge, of the beam. The concrete below the neutral axis $y\ y$ is usually so filled with very fine cracks that all the tension must be carried by the steel alone.

3. The principles just described apply to the reinforcement of concrete beams. By the addition of steel reinforcement to concrete in compression, such as columns, the bending and shearing stresses created in these members are resisted, and they are capable of carrying greater loads than plain concrete having the same proportion of ingredients.

In ordinary reinforced-concrete column construction, merely vertical rods are employed. They are tied, however, at intervals with wire or other ties. Concrete reinforced in this manner is considerably strengthened, because the longitudinal rods take up the flexure, or bending stresses, and the wire ties, or binders, prevent these rods from spreading under stress and also act as binders for the concrete.

4. In the design of reinforced-concrete construction, another principle is that applied to *hooped columns*, as they are called. This principle is best explained as follows: It is well known that a column of sand will not resist compression, since it will spread, and it is likewise certain that a cylinder of, say, very thin metal will sustain only a small load. However, if the cylinder is filled with sand, the combination, in which the tensile strength of the cylinder is realized with the compressive resistance of the sand, will result in a strong column, or post, capable of resisting considerable compression. This principle is applied to the reinforcement of concrete columns by binding, or tying, together the concrete with cylindrical hoops, or helical, or spiral, windings of steel.

5. Structural Elements of Reinforced-Concrete Construction.—The structural members of reinforced concrete are beams and columns, together with cantilevers and brackets.

In section, the beams are either rectangular, as shown in Fig. 2 (*a*), or **T**-shaped, as shown in (*b*). The rectangular beam is used as a lintel, or as a support for tanks in places where a floor slab cannot be combined with it. This type of beam is also used in the construction of elevator shafts, bulkheads, and light wells, as in such places a **T** section would be destroyed by cutting away the floor slab on each side. Such beams are always reinforced at the bottom, and if they are overstressed at the top, they are reinforced at this point also, so as to provide additional compressive strength. When reinforced in this manner, they are known as *double reinforced rectangular beams*.

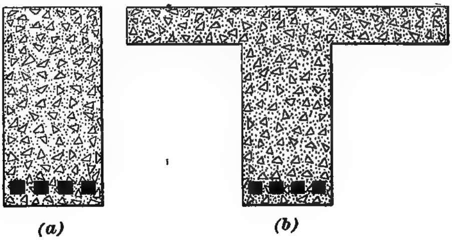
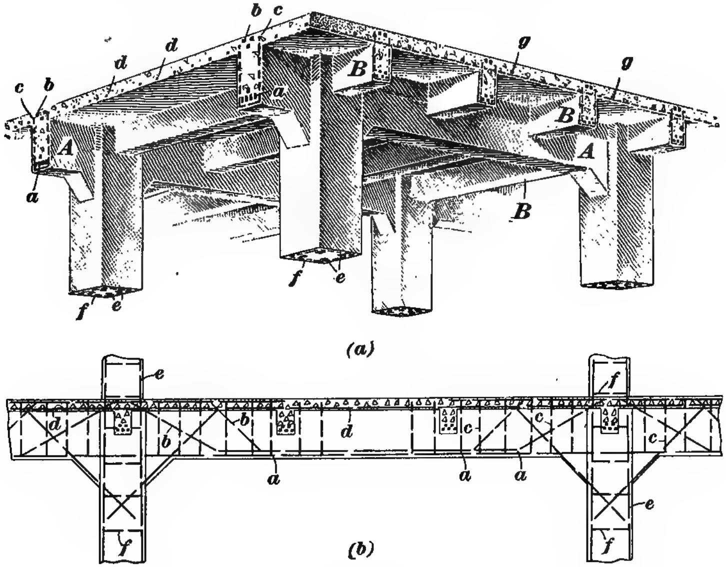

(a) (b)

FIG. 2

In the **T**-shaped section, the width of the beam at the top, that is, the width of the floor slab, is of course variable, and depends on the distance apart the beams are located in the floor and various other details. This type of beam, which is common in all building construction of reinforced concrete, is known as a *beam of* **T** *section.* It is the most economical form in which reinforced concrete is applied to resist transverse stress.

The reinforced-concrete column may be of any convenient section, either square, octagonal, or round; or, if used as a wall pier, it may be rectangular. Cantilevers are used extensively in reinforced-concrete work for carrying balconies, overhanging cornices, floors, and platforms; they are also used in cantilever foundation construction. Brackets are used in conjunction with columns to secure greater rigidity, to act as wind bracing, and to shorten the span of the beams or girders.



(a)

(b)

Fig. 3

These structural elements are applicable to any class of construction, and, together with reinforced-concrete arches used for bridges, and reinforced-concrete tanks subjected to bursting pressures, they embody the structural elements of all other kinds of structures.

6. Parts of Steel Reinforcement Defined.—In Fig. 3 (*a*) is shown a perspective view of a complete bay of a reinforced-concrete floor system, and in (*b*), a diagrammatic representation of a typical system of reinforcement for a concrete girder and column. In (*a*), the heavy members *A* running between columns are commonly known as **girders,** and the lighter members *B* running between girders, as **beams.** In both (*a*) and (*b*), the rods, or bars, *a* are the **main reinforcing bars,** or **rods,** of the girders. The beams, of course, have similar main reinforcing bars. Of these main reinforcing rods, several are bent up, as at *b*, to form **trussed bars.** The web reinforcement of the girders is shown at *c*, and consists of **U**-shaped pieces of iron or steel, called **stirrups.** The rods that reinforce the slab of the reinforced-concrete floor system, called **slab rods,** are shown at *d*. This slab reinforcement may consist of straight rods, expanded metal, woven-wire lath, or any other metallic reinforcement. The rods of the columns, shown at *e*, are called **longitudinal column rods,** and the looped separators, or ties, shown at *f*, **column ties.**

Any rod, or bar, used in reinforced-concrete construction to resist shearing stresses is usually designated as a **shear bar.** A rod, or bar, used to resist the shrinkage of the concrete in setting, or to provide against cracks due to thermal changes, is called a **shrinkage rod.** Shrinkage rods are shown at *g* in view (*a*). A rod used to connect abutting beams or girders, is called a **tension bar** or a **tie-bar.** The short rods used at the splice when longitudinal column rods are butted are called **splice rods,** or **bars.**

7. Methods of Failure of Reinforced-Concrete Beams.—To understand the purposes of the several types of steel reinforcement, it is necessary to know the different ways in which reinforced-concrete beams are liable to fail.

A reinforced-concrete beam, whether rectangular or of **T** section, may fail by transverse stress, which is caused either by the failure of the concrete at the top, due to excessive compressive stress, or by the failure of the steel, due to insufficient tensile resistance. In the latter instance, the steel may fail by breaking at the center of the span, or there may not be enough steel at another section and the steel may fail there, although there may be an excess of this material at the center of the span. However, in a reinforced-concrete beam containing much steel, the possibility of failure by the breaking of the steel at any point is remote, and very few, if any, failures of this kind are known. The failure of a concrete beam reinforced with metal bars may also be caused by the steel reinforcement slipping, or loosening its *bond* in the concrete.

All beams of reinforced concrete are subjected to both vertical and horizontal shear, and they are liable to fail by a combination of vertical shearing stresses and longitudinal tensile stresses. This form of failure is very common, as is evident on investigating the causes of nearly all the reinforced-concrete failures that have occurred.

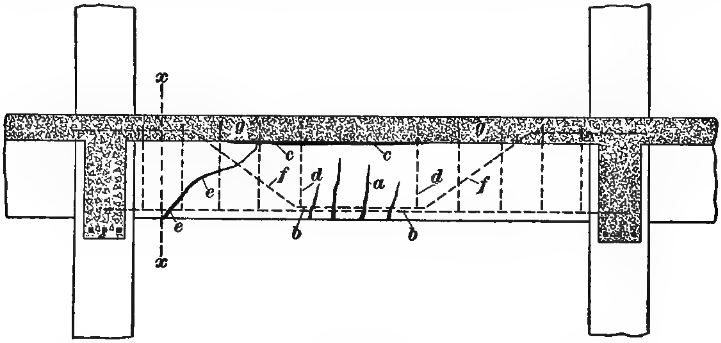


FIG. 4

Beams of **T** section may fail by longitudinal shearing at the junction of the beam section with the slab. However, this form of failure is due to a known cause. If the beam form is filled with concrete and allowed to set before the slab concrete is put in place, such a procedure leaves a plane of weakness, or a line of cleavage, between the junction of the beam section and the slab. In this method of failure, the concrete at the abutments tends to form a corbel, and the beam, with its load, tends to pull the main reinforcing bars away from this corbel.

8. Members to Resist Lines of Failure.—In Fig. 4 are illustrated a typical beam of T section, having the usual type of steel reinforcement, and the several methods of failure that might occur. At *a* are shown cracks, or lines of failure, that would be caused by lack of tensile resistance in the main reinforcing rods *b*. These cracks, although usually invisible, generally extend from the bottom surface to the neutral axis. They are nearly always present in concrete, but, of course, so long as the steel holds, the beam will not fail.

If the main reinforcing rods do not extend to the bearings, failure by vertical shear may occur near the abutment, along the line *x x*. Ordinarily, however, failures of this kind seldom happen, because the main rods usually extend across all such lines of vertical shear, and add greatly to the shearing resistance of the beam.

In case the slab concrete is not placed at the same time that the concrete of the beam section is poured, failure by shearing usually occurs at the junction of the beam with the slab, as shown at *c c*. The shearing resistance at this junction may be increased, however, by extending stirrups *d* into the slab. If the crack *c c* opens, it usually joins with a crack like *e e* at each end of the beam, as suggested in the preceding article.

The lines of failure indicated at *e e* are those which usually occur from diagonal tension stresses that cross these lines of failure at right angles. A beam is held against failure in this manner by placing stirrups in the concrete either vertically or obliquely. The bending up of the main reinforcing rods to form the trussed bar, as shown at *f f*, will also assist in resisting such stresses, and, besides, will provide against negative bending moment where tension instead of compression is created at *g g*. The line of fracture shown at *e e* is typical of nearly all reinforced-concrete failures.

9. Proportioning the Sectional Area of Steel Reinforcement.—In showing the sizes of steel reinforcement necessary for reinforced-concrete work on the estimating plans or working drawings, it is customary, where a particular type of bar is to be used, to indicate the size and the number of bars that are to be employed in the reinforcement of each beam, girder, or slab by some conventional abbreviation. If special deformed bars are to be used, the sizes must conform to the standard sizes of bars rolled by the manufacturer. If, however, the type of reinforcement has not been predetermined, the sectional area of the steel required for each structural element should be noted in square inches, and it is usually stipulated in the specifications or on the drawings what unit stress has been used in determining the areas and whether the reinforcement is to be of a mild, medium, or high-carbon steel. In bidding on the work, the manufacturers of special deformed bars must supply reinforcement at least equal to the area indicated on the plans.

If square twisted or round bars are to be used, the sectional areas may be given on the drawings and the selection of the size of bars left to the contractor; or, the number and the size of the bars can be given for each beam or girder. In case the sectional area is given, it is sometimes necessary to use several sizes of bars in one structural element in order to equal, and not materially exceed, the sectional area required. For example, assume that a girder is marked on the drawings as being 24 in. × 36 in. and having metal reinforcement of 8 square inches. For the reinforcement of this girder, eight 1-inch square rods would give the exact sectional area necessary, and as the sectional area of a 1⅛-inch diameter round rod is .99 square inch, it would take practically eight rods of this size. But if the girder were marked as requiring 4.4 square inches of metal reinforcement, then it would be necessary to use, say, four ⅞-inch round rods and two 1-inch square rods, the ⅞-inch rods having a total sectional area of 2.4 square inches, making a total of 4.4 square inches.

If the exact sectional area of the metal required is indicated, it is sometimes difficult for the contractor to supply the exact amount of metal by using standard sizes of rolled bars. In such cases, it is well for the designer, in stipulating the sectional area required on the drawings, to consider whether or not it can be made up by using standard round or square bars, provided bars of these sections are to be used.

METALLIC REINFORCEMENT

CHARACTERISTICS OF METALS USED FOR REINFORCEMENT

10. The physical character of the metals used for the reinforcement of concrete is a matter with which the engineer must be familiar if an efficient and economical design is to be produced.

While there are only the two materials, namely, *iron* and *steel*, employed commercially in the reinforcement of concrete, there are three grades of steel used, namely, *mild*, *medium*, and *high carbon*, all being produced by varying the chemical composition of the material. The grades known as soft, or mild, and medium are most used in the reinforcement of concrete, and these two steels are standardized by the specifications for structural steel adopted by the Association of American Steel Manufacturers. The grade called high carbon has a higher tensile resistance and a greater elastic limit than either soft or medium steel, and can be used to advantage in the construction of buildings in which the floors are not likely to be subjected to sudden shock or severe vibration.

The characteristics of steel are also altered by subjecting the metal to purely mechanical processes, such as *cold drawing* and *cold twisting*. The former process consists in pulling, or drawing, a metal rod, or bar, through dies to form wire, or rods of smaller diameter, while the latter consists in twisting a bar, or rod, of metal while cold to such an extent as to reduce its elasticity and raise its tensile strength.

11. Wrought Iron.—As a rule, **wrought iron** is employed as a reinforcement for concrete only in localities near a rolling mill in which this material is made and, consequently, where it may be obtained at a less cost than steel.

Iron as a metallic reinforcement of concrete has no advantages over steel; in fact, steel possesses greater strength and elasticity, is more extensively manufactured, and may be obtained ordinarily at less cost. Either round, square, or flat bar sections are used for the reinforcement of concrete; however, on special order it is possible to get square twisted bars.

Commercial wrought iron ordinarily has an ultimate tensile strength of from 48,000 to 50,000 pounds per square inch, and an elastic limit of from 22,000 to 24,000 pounds, with an elongation in 8 inches of from 8 to 12 per cent. A commercial bar of wrought iron should bend when cold without showing failure, through 180° around a diameter twice that of the test piece.

12. Mild, or Soft, Steel.—The commercial **mild,** or **soft, steel,** which is an excellent material for reinforcing rods, especially in concrete structures subjected to sudden strain or shock, should have a unit ultimate tensile strength of from 52,000 to 62,000 pounds and an elastic limit of not less than one-half this amount. It should have an elongation of at least 25 per cent. in 8 inches, and should be capable of being bent cold through 180° and hammered flat on itself without evidence of fracture on the outside circumference of the bend.

13. Medium Steel.—The grade of steel known as **medium** more fully meets all the requirements of reinforced-concrete construction than any other grade because it has a sufficiently high ultimate strength and elastic limit. Since medium steel is of a standard grade, it is more reliable than those steels known under the name of high carbon. Besides it is not so brittle nor so liable to fracture when bent cold—a great advantage where the steel is to be bent at the site of the operation. This kind of steel when used as reinforcement allows a safe unit tensile stress of from 16,000 to 18,000 pounds in the calculation for the sizes of the reinforcement, and the use of this safe unit stress gives a factor of safety of about 4.

Commercial medium steel should have an ultimate unit tensile stress of 60,000 to 70,000 pounds, and an elastic limit of not less than one-half this amount. Upon testing, the elongation should be found to be at least 22 per cent. in a length of 8 inches, and it should withstand bending through 180° around a diameter equal to the diameter or the thickness of the pieces tested. Such a test piece should not show fracture on the outside circumference of the bent portion.

14. High-Carbon Steel.—The term **high-carbon steel** does not definitely fix the quality of this material, owing to the fact that it is not included in the standard specifications of the Association of American Steel Manufacturers. This term is applied to steels rolled particularly for the reinforcement of concrete. Such steels contain a higher percentage of carbon than either medium or soft steel. The product as a rule is brittle, and possesses a unit ultimate tensile strength of from 80,000 to 100,000 pounds, with an elastic limit of about one-half of its ultimate strength. The amount of elongation in 8 inches is sometimes as low as 5 per cent., and seldom exceeds 15 to 18 per cent. Ordinarily, it will not bend much beyond a right angle without showing fracture. On account of the brittleness and general unreliability of high-carbon steel, the material should be carefully tested and inspected before being used in reinforced-concrete construction. For large operations, rather than purchase it in the open market, high-carbon steel should be rolled to special order.

The primary purpose in the use of high-carbon steel is to take advantage of the high elastic limit of the material, as in the design of reinforced concrete the real factor of safety is the ratio that the working stress bears to the elastic limit of the material. If the elastic limit of the steel is exceeded, the concrete will crack along the lower part of the beam, and besides showing this sign of distress, its shearing resistance will be weakened and the concrete will be liable to crush out at the top. It can readily be seen, therefore, that a steel having an elastic limit of 50,000 pounds possesses an advantage over one with elastic limit of only 30,000 pounds, and while the brittleness of this steel would cause it to be discarded for structural purposes, when embedded in concrete, sudden strains and shocks are to some extent absorbed by the mass of the monolith, and the safety of a brittle steel is not so much endangered as it would be if used for steel construction.

On account of its brittleness and liability to break on bending cold, high-carbon steel should not be used in the usual building operations as ordinarily conducted. But where the steel is carefully inspected and tested, a greater floor load may be carried with the same sectional area by using high-carbon instead of mild or medium steel.

15. Cold Twisted Steel.—If steel bars are drawn cold into smaller bars or wires, or if they are twisted, or subjected to torsion, their tensile strength and elastic limit will be increased. The percentage of increase in strength or elastic limit depends on the amount of stretching, drawing, or twisting to which the bar is subjected, and on the amount of deformation of the bar.

There are to be had on the market twisted bars of square and hexagonal section. Such bars are from 8 to 25 per cent. stronger than the bars in their original form. The percentage of increase of strength is greatest with the bars of small section and least with those of large section.

16. Rerolled Bars.—Bars made from old steel rails can be obtained for reinforcing concrete. In making such bars, the flange, web, and bulb of the rails are cut apart, and rerolled into square and round sections. The square sections may be had twisted, the twisting being performed while the material is hot.

Much of this material has a high elastic limit and tensile strength, but it is inclined to be brittle and ununiform in strength and ductility. Besides, any lapping or folding of the original material when passed through the rolls is liable to develop laminations throughout the bar, or rod, tending to diminish its homogeneity and, consequently, its strength.

This material should be employed only where it is possible to have a rigid inspection, and the physical properties of every lot of material should be determined by testing before being used in reinforced-concrete members of structural importance.

TYPES OF STEEL REINFORCEMENT

PLAIN BAR IRON

17. The cheapest form of metallic reinforcement for concrete is the **plain, round, rolled bar.** These bars can be obtained in any part of the United States, and as usual sizes are always found in the stock of mills or brokers, they may be secured at short notice. The pound price of round rolled bars is lower than that of any other form of rolled steel, so that with loose-bar systems of reinforced-concrete construction, they offer the cheapest and most available material. Ordinarily, the sizes used for reinforced-concrete construction vary from ⅜ to 1⅛ inches in diameter, the stock sizes changing by sixteenths or eighths. For slabs, ⅜- to ½-inch round bars are used, while for beam, girder, and column reinforcement, from ⅝- to 1⅛-inch bars are ordinarily employed.

The principal objection to the use of plain, round bars in reinforced-concrete work is that they do not have so much *bond* as deformed bars; that is, they are not gripped, or held, so well by the concrete. The rods are secured against slipping only by the adhesion between the concrete and the metal surface.

The advocates of the deformed bar for reinforced-concrete construction maintain that in reinforced-concrete floor systems subjected to vibration, plain rods tend to slip, and in time the bond between the rods and concrete is destroyed; they also claim that the bond between the concrete and the plain rods, or bars, is weakened when the concrete is saturated with moisture. However, the practical importance of these objections remains yet to be successfully proved,

TABLE I

AREAS AND WEIGHTS OF SQUARE AND ROUND BARS

Size Inch	Square ■		Round ●		Size Inches	Square ■		Round ●	
	Area Inch	Weight per Foot Pounds	Area Inch	Weight per Foot Pounds		Area Inches	Weight per Foot Pounds	Area Inches	Weight per Foot Pounds
1/16	.0039	.013	.0031	.010	1 1/16	1.1289	3.838	.8866	3.014
1/8	.0156	.053	.0123	.042	1 1/8	1.2656	4.303	.9940	3.379
3/16	.0352	.120	.0276	.094	1 3/16	1.4102	4.795	1.1075	3.766
1/4	.0625	.213	.0491	.167	1 1/4	1.5625	5.312	1.2272	4.173
5/16	.0977	.332	.0767	.261	1 5/16	1.7227	5.857	1.3530	4.600
3/8	.1406	.478	.1104	.376	1 3/8	1.8906	6.428	1.4849	5.049
7/16	.1914	.651	.1503	.511	1 7/16	2.0664	7.026	1.6230	5.518
1/2	.2500	.850	.1963	.668	1 1/2	2.2500	7.650	1.7671	6.008
9/16	.3164	1.076	.2485	.845	1 9/16	2.4414	8.301	1.9175	6.520
5/8	.3906	1.328	.3068	1.043	1 5/8	2.6406	8.978	2.0739	7.051
11/16	.4727	1.607	.3712	1.262	1 11/16	2.8477	9.682	2.2365	7.604
3/4	.5625	1.913	.4418	1.502	1 3/4	3.0625	10.413	2.4053	8.178
13/16	.6602	2.245	.5185	1.763	1 13/16	3.2852	11.170	2.5802	8.773
7/8	.7656	2.603	.6013	2.044	1 7/8	3.5156	11.953	2.7612	9.388
15/16	.8789	2.989	.6903	2.347	1 15/16	3.7539	12.763	2.9483	10.024
1	1.0000	3.400	.7854	2.670	2	4.0000	13.600	3.1416	10.681

as many buildings have been built with plain, round rods and are successfully resisting the loads and stresses to which they are subjected.

Plain square and flat bars are sometimes used for the reinforcement of concrete, though, generally, both of these sections, when so used, are deformed by twisting.

In the nomenclature of reinforced concrete, round, rolled sections are designated as **rods;** square sections, as **bars;** and rectangular sections, as **flats,** or **flat bars.**

18. In Table I are given the areas and weights of square and round bars up to and including 2 inches. Sizes larger than 2 inches are seldom used in reinforced-concrete construction. As a considerable percentage of such work is constructed with either round rods or plain square or twisted square bars, this table will be found useful in figuring the reinforcement.

19. Standard rolled bars varying by sixteenths of an inch are not readily obtainable except in the smaller sizes. If the larger sizes are required to the sixteenth inch, as, for example, a $1\frac{3}{16}$-inch bar, they must, as a rule, be especially rolled; consequently, they are more costly.

The pound price of rolled steel bars depends on the size of the section, and while the actual price fluctuates with the market, the variation in price between the several sizes is arranged on a base price agreed on by the manufacturers.

BARS OF SPECIAL CONSTRUCTION

20. Early Shapes.—In the effort of engineers and inventors to obtain bars of metal that would comply with the requirements of reinforced-concrete work, many types of special bars were evolved. Most of these bars are of the composite type—that is, they consist of bars made up of several pieces of metal—though there are several indented bars, twisted bars, and bars of unusual section that have been used more or less.

One of the first types of bars used for reinforced concrete is the **Hyatt bar.** This bar is illustrated in Fig. 5 and consists of flat bar iron or steel punched at intervals with round holes. In them are inserted short, round bars, which provide the necessary key, or bond, in the concrete work. These round bars can pass loosely through the hole in the flat bar and can be extended to form cross-bars for slab reinforcement.

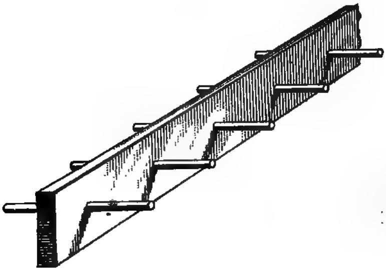


FIG. 5

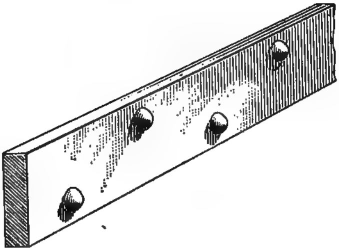


FIG. 6

A flat deformed bar was also proposed and used by the inventor of the Thacher bar. This bar, as shown in Fig. 6, consists of a flat bar of iron or steel, through which rivets are driven at intervals. The rivet heads furnish the projection necessary to increase the bond of the bar.

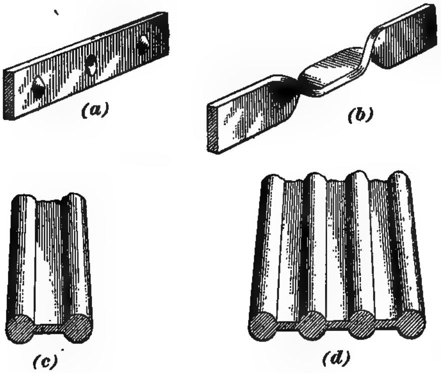


FIG. 7

In another bar, known as the **Klett bar,** angle clips are riveted to the flat bar. These clips are placed at intervals and answer the same purpose as the round bars or rivets in the Hyatt or the Thacher system.

21. Among the special deformed bars that have been used, though they are generally being superseded by more practical sections, are the *Staff*, the *DeMann*, and the *Unit bars*, the latter consisting of the Quad and the Siamese.

In Fig. 7 (*a*) is shown the **Staff bar.** It consists of a flat bar, through which a countersunk punch has been partly driven, thus forcing the metal out on the opposite side so as to form a deformation.

The **DeMann bar,** shown in Fig. 7 (*b*), is a plain, flat bar with a half twist at intervals.

The **Unit bar** is also shown in Fig. 7, the bar in (*c*) being known as the *Siamese* and that in (*d*), as the *Quad.* These bars consist of a flat rolled section combined with a round section, or bulb, the purpose being to provide in the round portion of the section enough area for the tensile stress, and to furnish a flat web that can be sheared partly and bent up into stirrups, or fins, for keying and bonding in the concrete, something after the manner of the Kahn bar to be described later. The Quad section contains four bulbs, or round sections, and the Siamese two. These bars were rolled in sections designated as ⅝, ¾, and ⅞ inch up to 1¼ inches, the dimension referring to the diameter of the round section. The webs were made from ⅛ to ¼ inch in thickness, as the rolls in forming the bars were kept farther apart so as to increase the section.

22. Square-Twisted Bars.—One of the most practical types of deformed bar is the **square-twisted reinforcing bar.** This bar consists of a square bar that is twisted by giving it a certain number of turns around its axis, either while it is hot or while it is cold. Formerly, cold square-twisted bars for reinforced-concrete construction were controlled by the patents of E. L. Ransome, taken out in 1884, and since then they have been known as the **Ransome bar.** The patents on this bar have expired, and they are now being placed on the market by numerous steel manufacturers.

By twisting the bar to the screw, or auger, shape, as shown in Fig. 8, a form is obtained that has great resistance to pulling from a mass of concrete. This is due to the fact that before the bond between the metal and the concrete can be destroyed, the concrete must be sheared.

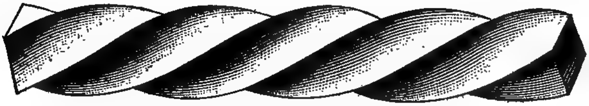


FIG. 8

If the square bars are twisted cold, their elastic limit and ultimate strength are increased from 8 to 25 per cent., and as the bars are thus given an initial strain, they offer advantages for reinforced-concrete work in that, under working loads they are subjected to a stress below their new elastic limit, so that the concrete is not likely to show hair cracks when the beams or girders are overloaded.

TABLE II

PHYSICAL PROPERTIES OF THE RANSOME BAR

Size of Bar Inches	Number of Twists per Foot of Length	Area of Bar Square Inches	Weight per Foot Pounds	Elastic Limit Pounds per Square Inch	Ultimate Tensile Strength Pounds per Square Inch	Elastic Limit of One Bar Pounds	Ultimate Tensile Strength of One Bar Pounds
¼	5	.0625	.213	62,350	86,700	3,897	5,419
⅜	3	.1406	.478	61,800	86,600	8,689	12,176
½	2	.2500	.850	60,120	86,850	15,030	21,713
⅝	2	.3906	1.328	57,890	85,820	22,612	33,520
¾	1	.5625	1.913	56,720	85,240	31,905	47,948
⅞	1	.7656	2.603	56,150	84,730	42,988	64,869
1	¾	1.0000	3.400	55,760	84,275	55,760	84,275
1¼	½	1.5625	5.312	55,450	83,150	86,641	129,921

The Ransome square-twisted bars are typical of most of the cold square-twisted steel bars on the market, so that Table II, which gives the sizes, number of twists to the foot in length, the weight, the elastic limit, and the ultimate strength of the bar, may be considered as applying to cold square-twisted bars in general.

23. As stated in Art. **16**, it is the practice with some manufacturers to cut up and reroll steel rails into sections suitable for reinforcing concrete, and usually these are square sections that are twisted while the metal is hot. There are also manufactured square hot-twisted bars made from the original billet. Such bars are usually of mild or, more frequently, of medium steel, and are reliable because they are uniform in strength and ductility. Hot-twisted, high-carbon steel bars with a high elastic limit and great ultimate strength are also to be had, especially in the smaller sizes. But such bars should be carefully inspected and tested, and those which show excessive brittleness should be discarded.

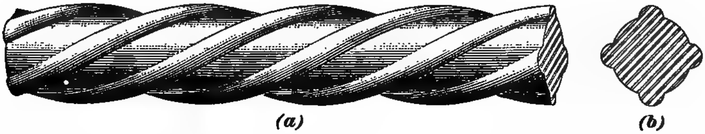
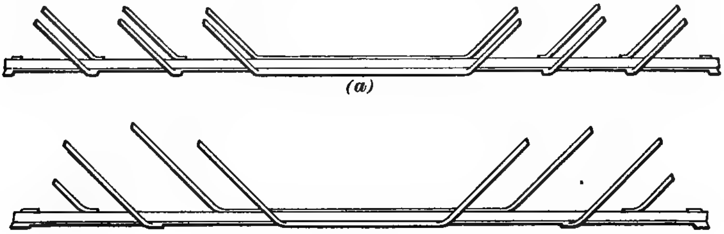


FIG. 9

24. Square-twisted bars cost very little more than plain round bars, and may be obtained in all commercial centers and ordinarily in sizes ranging by eighths. The additional cost for cold-twisted square bars depends on the number of bars that must be handled for a given weight, and, consequently, the smaller bars cost the most. Usually, the cost of twisting is estimated by adding a certain amount to the market price per pound of the plain bars. For instance, $\frac{1}{4}$-inch square Ransome bars cost $\frac{8}{10}$ cent more per pound than plain square bars, while the same bars $\frac{3}{4}$ inch square cost $\frac{3}{10}$ cent per pound in excess of plain bars. Intermediate sizes vary between these limits.

25. Spiral Bar.—In Fig. 9 (*a*) is shown a type of twisted bar for reinforced-concrete construction that partakes of the nature of the square-twisted bar. This bar is called the **spiral bar,** and it is made by rolling steel to the section shown in (*b*) and then twisting it. High-carbon steel having elastic limits of from 55,000 to 70,000 pounds per square inch is used in these bars. Thus, the twisted bars have elastic limits of from 65,000 to 80,000 pounds.

From the illustration, it will be observed that the section of the bar is practically round, with four attached half-round beads. These beads assume the spiral form on twisting the bar, and the same advantage with regard to the bond is secured as with square-twisted bars. An added advantage is assumed to exist in the elimination of all the sharp corners that occur in the square bar. The tendency of the concrete to crack in setting, as it sometimes will do at the sharp edge of a bar, is therefore overcome.


(*a*)

(*b*)

Fig. 10

26. Kahn Trussed Bar.—Fig. 10 shows two styles of a deformed bar known as the **Kahn bar.** In the old style of bar, shown in (*a*), the prongs are opposite each other, while in the new style, shown in (*b*), they are staggered. The shape of the bar in section, as shown in Fig. 11, is either square or of a special section, with a fin projecting on each side coincident with one of the diagonals of the square. These fins are partly sheared across and also in a direction parallel with the axis of the bar, and are bent up, as shown in Fig. 10, so as to form a grip, or key, with the concrete and to provide the stirrups, or web members, necessary to resist diagonal stresses. The bent-up portion of the bar is sometimes spoken of as a *shear member*, and the bar is frequently called the **Kahn trussed bar** from its resemblance to a truss frame.

These bars have come into extensive use and are made in the several sizes indicated in Fig. 11, which shows the sections

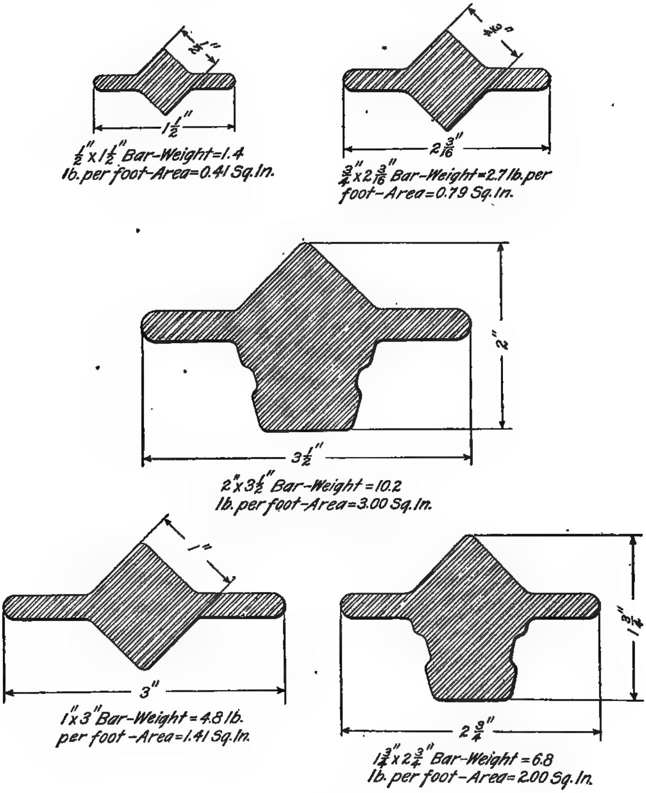
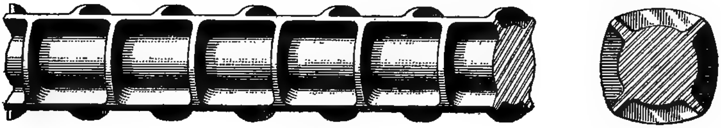

½"
1½"
½"x1½" Bar-Weight=1.4 lb. per foot-Area=0.41 Sq. In.
¾"
2 3/16"
¾"x2 3/16" Bar-Weight=2.7 lb. per foot-Area=0.79 Sq. In.
2"
3½"
2"x3½" Bar-Weight=10.2 lb. per foot-Area=3.00 Sq. In.
1"
3"
1"x3" Bar-Weight=4.8 lb. per foot-Area=1.41 Sq. In.
1¾"
2¾"
1¾"x2¾" Bar-Weight=6.8 lb. per foot-Area=2.00 Sq. In.

FIG. 11

to half their natural size. The respective sectional areas of the bars are also given in this figure. These bars are rolled of mild steel, as this metal will best stand the shearing stresses to which the bar is subjected in manufacture. The elastic limit of the metal in the bars ranges from 33,000 to 35,000 pounds per square inch.

The advantages claimed for the Kahn trussed bar, and these are substantiated in practice, are: (1) That they reinforce the concrete in a vertical plane as well as in a horizontal one; (2) that the reinforcement is so disposed as to be inclined to a vertical plane, so as to provide resistance against oblique tensile stress; (3) that the metal is distributed in close proportion to the stresses existing at any place, this being accomplished by leaving the entire section of the bar at the middle of the span; and (4) that the shear members of the reinforcement are a part of the horizontal reinforcement and are connected rigidly to it.

This type of bar is used for reinforcing beams, girders, and columns. It is also employed in a composite construction consisting of hollow tile and reinforced concrete, as described in another Section.



(a) (b)

FIG. 12

27. Kahn Cup Bar.—One of the latest types of reinforcing bars to be commercially used is the **Kahn cup bar** illustrated in Fig. 12 (*a*). The general section of this bar is round, with four ribs, or fins, at the ends of two diagonals, as shown in section at (*b*). These ribs are connected by cross-ribs, forming cups, or depressions, of such a shape as to allow the concrete to flow into them readily. In this way, a positive mechanical bond in the concrete is provided. It is claimed that by the process of its manufacture this bar is rolled, or worked, in such a manner as to develop a uniform fiber structure, giving increased elastic limit and ultimate strength without affecting its ductility.

The cup bar can be obtained in any reasonable length, and in nominal dimensions of section from ⅜ to 1¼ inches, varying by eighth inches. The sectional area of the cup bar corresponds with the area of square bars of the same dimensions. The standard nominal sizes of the cup bar, its weight per foot, sectional area, and average ultimate strength, together with its approximate elastic limit, are given in Table III.

TABLE III

PROPERTIES OF THE KAHN CUP BAR

Nominal Size of Bar Inches	Sectional Area Square Inches	Weight per Foot Pounds	Elastic Limit of Each Bar Pounds	Ultimate Tensile Strength of Each Bar Pounds
$\frac{3}{8}$	.1406	.502	5,000	9,800
$\frac{1}{2}$	.2500	.893	9,000	17,500
$\frac{5}{8}$	.3906	1.394	14,000	27,300
$\frac{3}{4}$	.5625	2.008	21,000	39,400
$\frac{7}{8}$	.7656	2.733	28,000	53,600
1	1.0000	3.570	37,000	70,000
$1\frac{1}{8}$	1.2656	4.518	47,000	88,600
$1\frac{1}{4}$	1.5625	5.578	58,000	109,400

28. Trus-Con Bar.—Another deformed bar made by patentees of the Kahn bar, is the type of bar illustrated in Fig. 13. This bar, known as the **trus-con bar,** is round in section and has notched shoulders for holding washers. The washers are punched so that they will slip over the projections, and, then by turning, they are locked with a bar. The trus-con bar is designed for reinforced-concrete construction in which a plain deformed bar without stirrups is required. They are made in $\frac{5}{8}$-, $\frac{3}{4}$-, 1-, $1\frac{1}{4}$-, and $1\frac{1}{2}$-inch sizes.

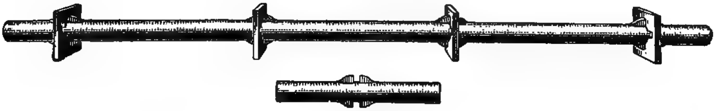


FIG. 13

29. Square-Twisted Lug Bars.—The **twisted lug bar** shown in Fig. 14 is a development of the square-twisted bar. The square bar in this case is rolled with rounded corners, and with projections, or lugs, at intervals, as indicated at *a*, thus providing additional mechanical bond in the concrete. The rounded corners of this bar eliminate the sharp angles of the ordinary square-twisted bar, and is thus assumed to provide against initial cracks that usually occur when concrete sets. These bars have a high elastic limit, and their safe working strength, which is based on a safe unit stress of 20,000 pounds, is given in Table IV. This table also gives their weight per foot and their net area in square inches.

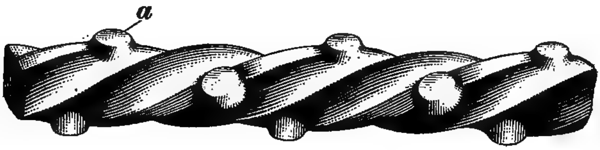


FIG. 14

TABLE IV

PROPERTIES OF TWISTED LUG BARS

Size of Bar Inches	Net Sectional Area Square Inches	Weight per Foot Pounds	Safe Working Stress for Each Bar Pounds
1/4	.0625	.222	1,250
3/8	.1406	.492	2,810
1/2	.2500	.870	5,000
5/8	.3906	1.350	7,810
3/4	.5625	1.940	11,250
7/8	.7656	2.640	15,310
1	1.0000	3.450	20,000
1 1/8	1.2656	4.350	25,310
1 1/4	1.5625	5.370	31,250

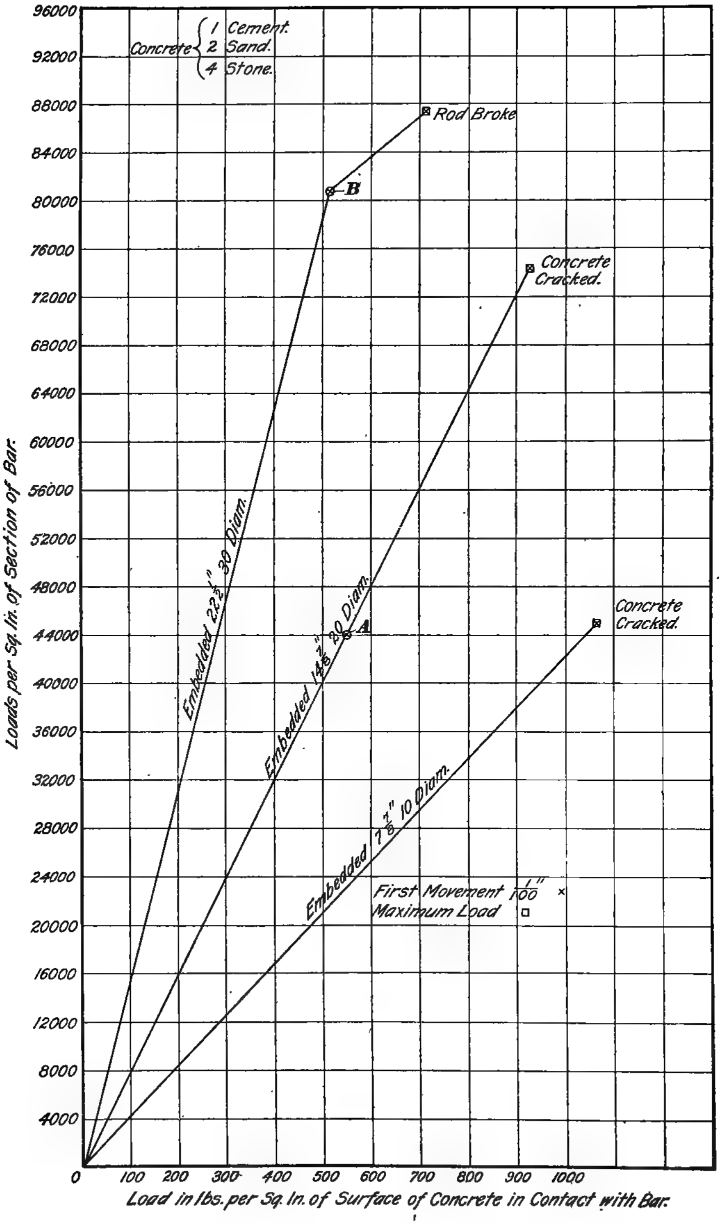

Concrete { 1 Cement. 2 Sand. 4 Stone.
Rod Broke
B
Concrete Cracked.
Concrete Cracked.
A
Embedded 22½" 30 Diam.
Embedded 14⅞" 20 Diam.
Embedded 7⅞" 10 Diam.
First Movement 1/100" ×
Maximum Load □
96000
92000
88000
84000
80000
76000
72000
68000
64000
60000
56000
52000
48000
44000
40000
36000
32000
28000
24000
20000
16000
12000
8000
4000
0
100
200
300
400
500
600
700
800
900
1000
Loads per Sq. In. of Section of Bar.
Load in lbs. per Sq. In. of Surface of Concrete in Contact with Bar.

Fig. 15

Mild steel having an elastic limit of not less than 32,000 pounds and an ultimate strength of from 55,000 to 65,000 pounds is used in rolling cold-twisted lug bars. By the cold-twisted process, the ultimate strength is raised to 84,000 or more pounds. This type of bar may be obtained, as will be observed from Table IV, in sizes corresponding in sectional area to from ¼- to 1¼-inch square bars, and in any length from 40 to 60 feet.

30. The results of tests made at the University of Pennsylvania to determine the bond of ¾-inch, cold-twisted lug bars is shown in Fig. 15. Three bars embedded to different depths in the concrete were tested. The rod embedded deepest broke before the concrete failed, while in the two other cases the concrete failed first. In the case of the rod embedded only 7⅞ inches, the concrete cracked and released the bar when the load on the latter was about 45,000 pounds per square inch. It will be noted, however, that the concrete did not fail until the force tending to make the bar slip was over 1,000 pounds per square inch of surface in contact, when the adhesion between the steel and the concrete was at last overcome. The bar that was embedded 14⅞ inches slipped slightly at the point *A* when the load tending to cause slip was about 550 pounds per square inch, but it regained its grip

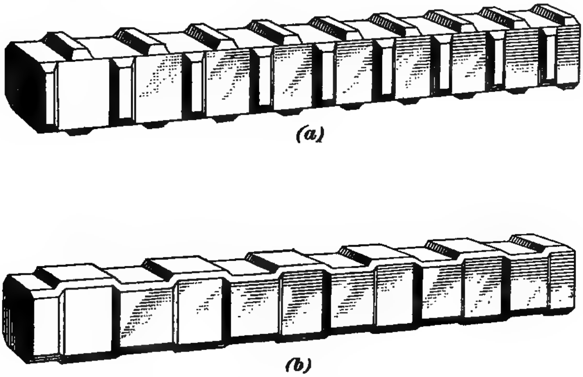


FIG. 16

immediately, and did not slip until the load tending to cause slip was over 900 pounds per square inch of surface in contact and the stress in the steel was over 74,000 pounds per square inch. The bar embedded 22½ inches slipped when the load tending to cause slip was over 500 pounds per square inch, as shown at *B*, and continued to slip during the remainder of the test until finally it broke when the stress in it reached almost 88,000 pounds per square inch.

These tests demonstrate two things: first, the high unit tensile strength of the steel (almost 88,000 pounds), and, second, that the concrete takes a very strong grip on this make of bar. In the first case given, the adhesion was over 1,000 pounds per square inch of surface in contact.

31. Corrugated Bar.—In Fig. 16 is shown a deformed bar of the corrugated type known as the **Johnson bar,** named after its inventor, A. L. Johnson. The old style of bar is shown in (*a*), and the new style in (*b*). It will be observed that the bar is rolled with a number of projections on its four sides and that these projections are spaced staggered on adjacent surfaces. High-carbon steel having an elastic limit of from 65,000 to 70,000 pounds per square inch is used in making the Johnson bar. Cold-bending tests of these bars frequently indicate brittleness.

This type of bar was one of the first deformed bars to be manufactured. The sides of the raised corrugations are

TABLE V

SIZE, NET SECTION, AND WEIGHT OF CORRUGATED, OR JOHNSON, BARS

Size of Bars Inches	Weight per Foot Pounds	Net Section Square Inches
¼	.78	.19
¾	1.56	.38
⅞	2.25	.55
1	2.90	.70
1¼	4.56	1.10

formed at such an angle with the surface of the bar that the concrete will not be liable to slip. The purpose in forming the sides of the projections of the bar at an angle is that clearance must be provided in rolling the bars.

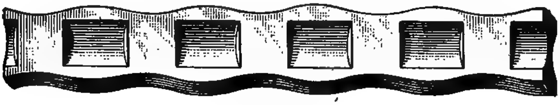


FIG. 17

Johnson bars are made in sizes from $\frac{1}{4}$ to $1\frac{1}{4}$ inches square, varying by eighth inches. The sizes and net sections, together with the weight per foot, are given in Table V.

32. Universal Bar.—In Fig. 17 is shown a type of flat deformed bar, called the **Universal bar,** that has been used to some extent for reinforcement. It consists of a flat bar that has indentations rolled in the flat sides. These indentations, besides furnishing deformations for bonding with the concrete, also supply means by which the bars may be spliced or by which stirrups may be secured to the bars. The construction of a splice connection for this type of bar is shown in detail in Fig. 18. As shown at *a*, a rectangular ring of metal is fitted over the splice bars; then, a wedge, or clip, *b* is placed in the indentation, and the ring is securely wedged in position. The connection made in this manner

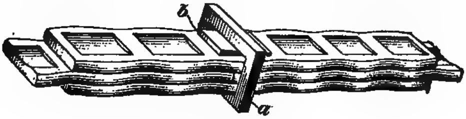


FIG. 18

offers great tensile resistance, and may be designed to equal the strength of the bar at the net section.

The Universal bar may be obtained in the sizes given in Table VI. This table also gives the sectional area for these standard sizes.

TABLE VI

NET SECTIONS AND WEIGHTS OF UNIVERSAL BARS

No.	Size of Bar Inches	Weight per Foot Pounds	Net Section Square Inch
1	$\frac{1}{4}\times 1$	.73	.19
2	$\frac{3}{8}\times 1\frac{3}{8}$	1.35	.41
3	$\frac{3}{8}\times 1\frac{3}{4}$	1.97	.54
4	$\frac{3}{8}\times 2$	2.27	.65
5	$\frac{3}{8}\times 2\frac{1}{2}$	2.85	.80

33. Thacher Bar.—The deformed bar illustrated in Fig. 19, called the **Thacher bar,** was devised in order to obtain a bar so deformed that the net section throughout the bar would be uniform. By having the section uniform, the bar is of the same strength at every point, and no unnecessary metal is used in making one part of the bar stronger than another. In forming this bar, an effort is made to eliminate all sharp corners, so that the embedment of the bar will be more perfect and its strength and bond increased. The Thacher bar is rolled from medium steel and has an elastic limit of about 35,000 pounds. It is made in a great variety of sizes, ranging from $\frac{1}{4}$ to 2 inches and varying by eighth inches. The sizes, areas, weights, and ultimate strengths of this type of bar are given in Table VII.

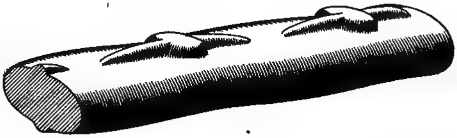


FIG. 19

34. Diamond Bar.—One of the most recent forms of deformed bars is the **Diamond bar,** which is illustrated in Fig. 20. This bar is rolled with a series of corrugations, or fillets, and in such a way that it has a cross-section of constant

TABLE VII

SIZE, WEIGHT, AND ULTIMATE TENSILE STRENGTH OF THACHER BARS

(*Medium Steel*)

Diameter of Bar Inches	Weight per Foot Pounds	Area of Net Section Inches	Average Ultimate Tensile Strength for Each Bar Pounds
1/4	.16	.047	3,000
3/8	.34	.10	6,400
1/2	.61	.18	11,500
5/8	.95	.28	17,900
3/4	1.39	.41	26,200
7/8	1.87	.55	35,200
1	2.41	.71	45,400
1 1/8	3.06	.90	57,600
1 1/4	3.74	1.10	70,400
1 3/8	4.49	1.32	84,500
1 1/2	5.30	1.56	99,800
1 5/8	6.15	1.81	115,800
1 3/4	7.07	2.08	133,100
1 7/8	7.99	2.35	150,400
2	9.01	2.65	169,600

area. Thus, all parts of the bar are of equal strength and no metal is wasted. The point of superiority claimed for this bar is that the bond, or grip, on it is uniform and continuous along its entire length. The manufacturers claim that the bar has a maximum efficiency with regard to bond and that there is no tendency for the bar to twist or turn, as is

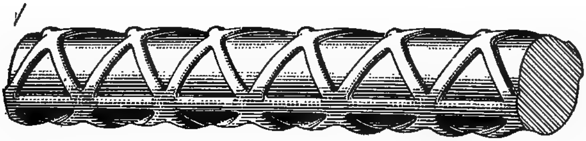


FIG. 20

the case with square-twisted bars. Likewise, all sharp edges, or corners, tending to split the concrete are eliminated. The diamond bar is made in the several standard sizes.

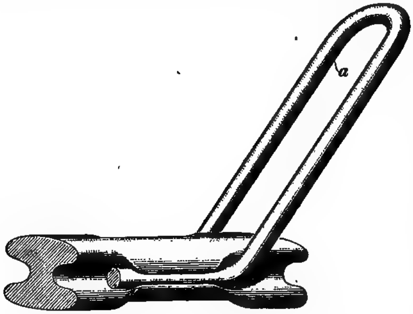


FIG. 21

35. Monolith Steel Bars.—In Fig. 21 is shown a peculiar form of reinforcing bar, known as the **monolith steel bar.** This type of bar has no sharp corners, and is not reduced in area or strength by deformations. It also has large surface area. The bar is grooved on the sides so that round iron stirrups, as shown at *a*, may be inserted. These stirrups are held in place in the bar by swedging the flanges of the bar together. This swedging of the flange serves to hold the stirrup in its grip and provides additional bonding resistance in the concrete.

The monolith steel bars are made in sections equivalent to $\frac{1}{2}$-, $\frac{5}{8}$-, 1-, and $1\frac{1}{2}$-inch square bars, and are rolled for stirrups of $\frac{3}{16}$, $\frac{5}{16}$, $\frac{3}{8}$, and $\frac{1}{2}$ inch diameter. The advantage claimed for this bar is that it can be easily assembled. There is, however, a possibility that concrete placed dry may not enter the grooves, or depressions, in the sides of the bars, thus causing the grip to decrease, but this can hardly occur if wet concrete is used.

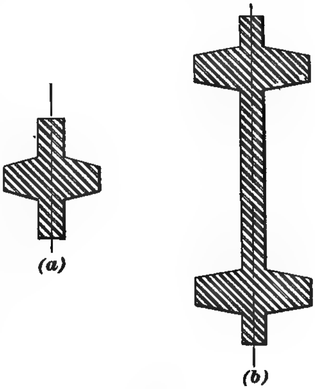

(a) (b)

FIG. 22

36. Columbian Bar.—A rolled shape known as the **Columbian bar** is extensively used in the construction of the Columbian fireproof floor systems and reinforced-concrete structures. The typical forms of this type of bar are shown in Fig. 22. Mild steel made by either the open-hearth or the Bessemer process is used for rolling the Columbian bar. This kind of steel has an ultimate strength of from 60,000 to 70,000 pounds per square inch and an elastic limit of one-half the ultimate strength. The smallest bar, shown in Fig. 22 (*a*), is known as the 1-inch bar, while the type of bar shown in (*b*), is made in 2-, 2½-, 3½-, and 4¼-inch sizes. The 5-inch bar carries a double rib at the bottom. The 1-inch bar may be used between the larger sections to make up sectional areas, and in order to provide against negative bending moment and to act as ties, it can be bent up to form truss bars.

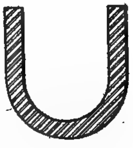


FIG. 23

37. U Bars.—In Fig. 23 is shown a type of **U bar** that can be used to advantage in reinforced-concrete construction either as a tension or as a compression member, being particularly efficient for the latter purpose. These bars are rolled from high elastic-limit steel, and are in some instances made from rerolled steel rails. They are ¼-, $\frac{5}{16}$-, ⅜-, and ½-inch in thickness, weigh from 3 to 9 pounds per foot, and can be obtained in lengths up to 60 feet.

U bars can be arranged for metallic reinforcement in concrete construction by fastening them together in the manner indicated in Fig. 24. Likewise, they can be arranged either singly or in combination to form sections efficient in resisting compression in the upper part of rectangular concrete beams, or in framed work of concrete, such as roof or latticed trusses.

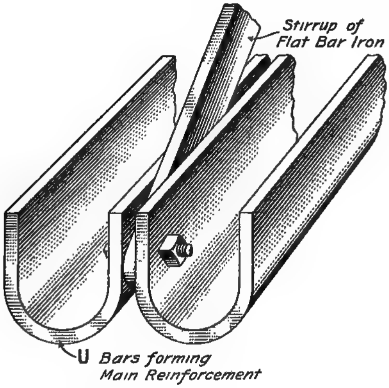


FIG. 24

STRUCTURAL SHAPES USED AS STEEL REINFORCEMENT

38. The usual rolled-steel structural shapes have been used extensively for reinforced-concrete construction. They have an advantage in that they can be readily obtained, but their cost per pound is greater than that of plain round or square-twisted bars.

Of these structural shapes, the most available sections for ordinary reinforced-concrete construction are small angles and tees. The tees, however, being more expensive, cannot be used in competition with angles or other cheaper rolled structural sections. Small **I** beams—that is, 4-, 6-, and 8-inch beams—have been used in several systems of reinforced-concrete construction, and where these shapes are employed, an effort is usually made to suspend the centering from the steel reinforcement.

All structural shapes, however, offer such large flat surfaces for embedment in the concrete that they tend to create lines of cleavage when the concrete sets, thus lessening the grip, or bond. There is no way of deforming such structural shapes without materially decreasing their net section, so that in order to provide additional bond or stirrups, considerable shop work is required.

Structural shapes as metallic reinforcement for concrete work are being superseded by either plain or deformed rolled bars. One advantage of the use of structural shapes is that a rigid framework can be built up, after the manner of skeleton construction, though of much lighter sections, and the concrete then filled in around it.

REINFORCEMENT FOR SLAB CONSTRUCTION

39. Expanded Metal.—Among the earlier forms of metallic reinforcement for concrete is the distorted, or deformed, steel plate known as **expanded metal,** a familiar illustration of which is shown in Fig. 25 (*a*). This form of reinforcement is manufactured by partly shearing a sheet of steel in parallel rows, as shown in Fig. 25 (*b*), and then pulling the material sidewise, thus forming a diamond mesh. In this way, the area of a sheet is increased about eight times, with a corresponding decrease in weight per unit area and without any waste of material. The steel from which expanded metal is manufactured has a unit tensile strength of about 65,000 pounds and an average elastic limit of about 30,000 pounds. It is principally used in reinforced-concrete construction as slab reinforcement, and is sometimes employed as a web to transfer the horizontal shear in reinforced-concrete beams from the metallic reinforcement through the web of the beam.

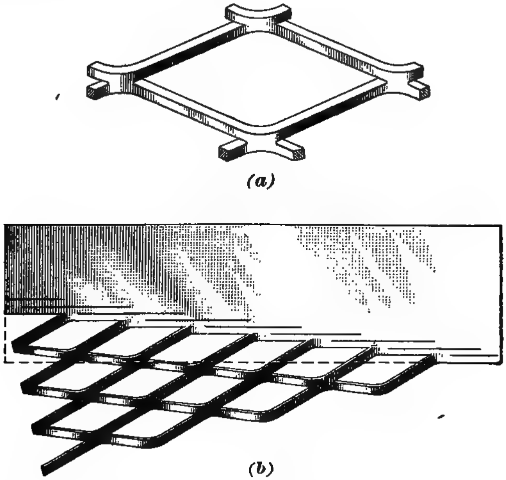


FIG. 25

Expanded metal is manufactured from sheets having gauge thicknesses of from No. 4 to No. 18 and meshes from ½ to 6 inches wide. Thus, there is a wide range of sizes and sectional areas.

The sizes of expanded-metal sheets as manufactured by expanded metal companies are given in Table VIII. This table also gives the sectional area in square inches per foot in width, and thus furnishes the necessary data for determining the strength of the metallic reinforcement when used for making slabs.

TABLE VIII

SIZES OF EXPANDED METAL

U. S. Standard Gauge of Sheet Steel	Sectional Area per Foot of Width Square Inch	Weight Pounds per Square Foot	Elastic Limit per Foot of Width Pounds	Length of Sheets Feet	Thickness of Metal Inch	Length of Mesh Inches	Width of Mesh Inches	Width of Separate Strands Inch
18	.203	.69	12,200	8	$\frac{1}{20}$	$1\frac{1}{2}$	$\frac{1}{2}$	$\frac{5}{64}$
12	.253	.86	15,200	8	$\frac{7}{64}$	2	$\frac{3}{4}$	$\frac{5}{64}$
12	.188	.64	11,300	12	$\frac{7}{64}$	3	$1\frac{1}{2}$	$\frac{7}{64}$
16	.087	.29	5,220	8	$\frac{1}{16}$	6	$2\frac{1}{4}$	$\frac{1}{8}$
16	.130	.44	7,800	8	$\frac{1}{16}$	6	$2\frac{1}{4}$	$\frac{3}{16}$
16	.059	.20	3,540	12	$\frac{1}{16}$	8	3	$\frac{7}{64}$
12	.109	.37	6,550	12	$\frac{7}{64}$	8	3	$\frac{1}{8}$
10	.162	.55	9,700	12	$\frac{9}{64}$	8	3	$\frac{9}{64}$
10	.243	.81	14,600	12	$\frac{9}{64}$	8	3	$\frac{7}{32}$
10	.324	1.07	19,400	12	$\frac{9}{64}$	8	3	$\frac{9}{32}$
7	.400	1.36	24,000	12	$\frac{3}{16}$	8	3	$\frac{13}{64}$
4	.245	.84	14,700	12	$\frac{15}{64}$	12	6	$\frac{1}{4}$
4	.368	1.26	22,100	12	$\frac{15}{64}$	12	6	$\frac{3}{8}$

Standard expanded-metal sheets are 8 and 12 feet in length, and may usually be obtained in widths of from 1 to 6 feet. Generally, they are shipped in bundles that contain five sheets wired together.

40. In placing expanded metal for reinforcing slabs, care should be taken to arrange the metal so that the tensile strains will lie in the same direction as the run of the bars of the mesh. In order to explain this statement reference is made to Fig. 26. At (*a*) is shown the incorrect method of placing expanded metal, while at (*b*) is shown the correct way. It will be noticed from view (*a*) that all the tensile stresses in the slab would be taken up by the tendency of one mesh to tear from the others at the points *a*, which are already seriously weakened by the shearing operation in the manufacture of the metal. From view (*b*) it will be observed that the bars of the mesh *b* offer their sectional resistance to the tensile stress. One failure in reinforced-concrete construction showed that the expanded metal used to reinforce the slabs was placed in the wrong manner, and was probably one of the causes of the collapse.

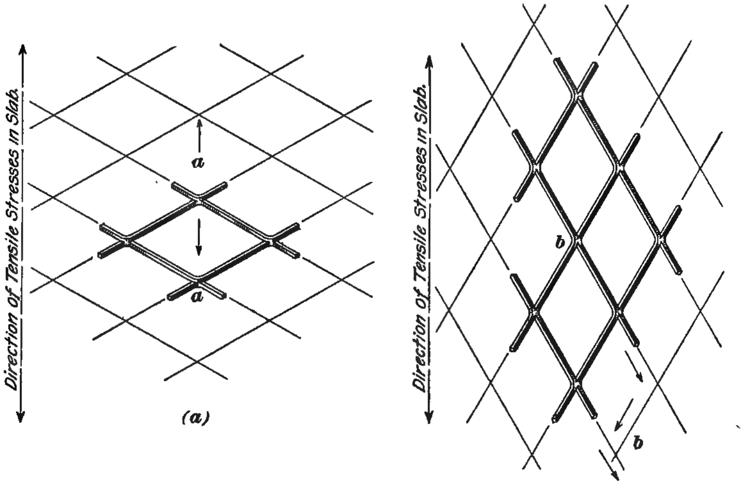

Direction of Tensile Stresses in Slab.
a
a
(a)
Direction of Tensile Stresses in Slab.
b
b
(b)

Fig. 26

It is customary to lap expanded metal about 6 inches, and such laps should always occur over beams, girders, or other points of support. It is not necessary to have much of a side lap if the meshes are strongly wired together, as the wiring provides the necessary shrinkage reinforcement. However, any additional side lap provides increased shrinkage resistance, which is desirable in reinforced-concrete slab construction.

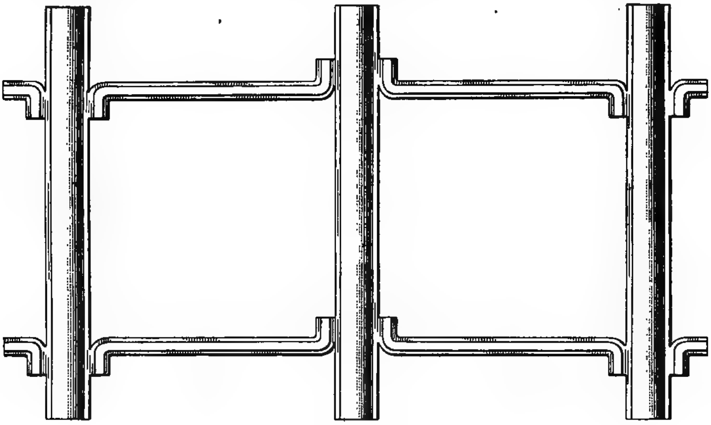


Fig. 27

41. Kahn Expanded Metal.—An excellent concrete-slab reinforcement, called **Kahn expanded metal,** is made from a section somewhat similar to the Kahn bar. This expanded metal, which has the form shown in Fig. 27, is manufactured by shearing a rolled section of the form illustrated in Fig. 28 along lines parallel with its length, and then pulling the metal so sheared in a direction at right angles to the lines of shear. In this type of expanded metal, the main bars of the section represent the material available for the reinforcement of the slab, while the light cross-bars act as

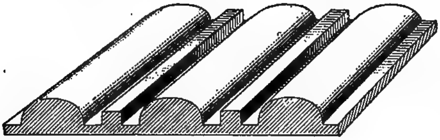


Fig. 28

spacing bars and also as shrinkage rods when embedded in concrete.

The advantage claimed for the Kahn expanded metal over those having a diamond mesh is that it transmits the load directly to the supports without any tendency to elongate or distort. Consequently, slabs reinforced with it are not liable to so much deflection as those reinforced with the diamond-mesh material. The cross-bars, or spacing bars, tend to transmit directly to the main ribs, all intermediate strains that are induced by temperature or shrinkage, and at the same time do not add materially to the weight of metal required. These secondary bars also insure the correct spacing of the main bars—a very desirable feature to be possessed by any slab reinforcement.

The Kahn expanded metal is so formed that the main reinforcing ribs always have the same section. Each rib has a sectional area of .08 square inch. The amount of metal required per foot of width of slab is controlled by varying the distance between ribs by making the cross-ribs longer or shorter as required and thus changing the spacing of the

TABLE IX

PROPERTIES OF KAHN EXPANDED METAL

No.	Distance Between Ribs Inches	Sectional Area of Metal per Foot in Width Square Inch	Safe Tensile Stress per Foot in Width Pounds	Ultimate Stress per Foot in Width Pounds	Width of Sheet Inches	Weight per Square Foot Pounds
2	2	.48	8,640	33,600	17	2.13
3	3	.32	5,760	22,400	25	1.43
4	4	.24	4,320	16,800	33	1.08
5	5	.19	3,420	13,300	41	.87
6	6	.16	2,880	11,200	49	.72
7	7	.14	2,520	9,800	57	.62
8	8	.12	2,160	8,400	65	.55

main ribs. It is thus evident that considerable range in the sectional area of reinforcement may be had, for in a 2-inch mesh, the main reinforcing ribs are placed 2 inches from center to center, while in the 8-inch mesh, the same bars are placed 8 inches from center to center.

The sectional area per foot in width of slab, together with the safe and ultimate tensile stresses for the different sizes of meshes, the weight per square foot, and other properties, are given in Table IX.

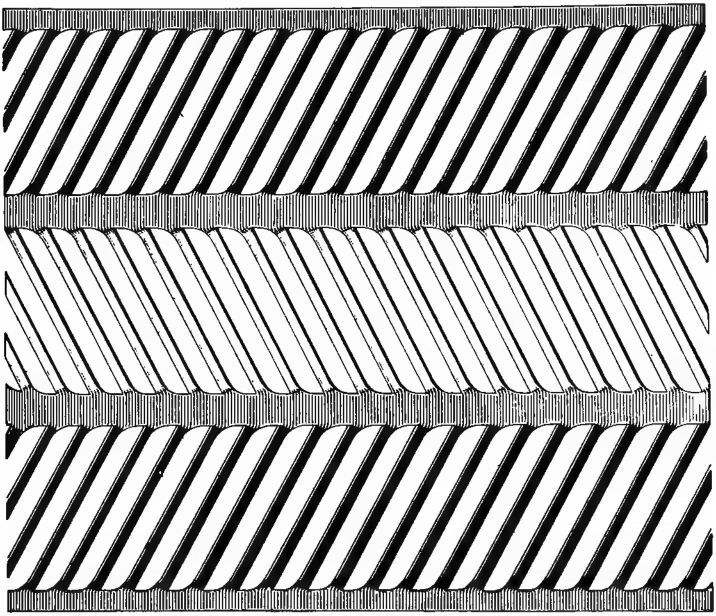


FIG. 29

In the table, both the safe tensile and ultimate stresses are the strength values for 1 foot in width of the expanded-metal sheets. Any of these sheets can be obtained in lengths of 12, 14, or 16 feet, and on special order they can be had any practical length.

42. Herring-bone Expanded Metal.—In Fig. 29 is shown a type of expanded metal known as **herring-bone**

metal lath. This metal is particularly interesting, because, when it is bent into the form shown in Fig. 30, it provides a self-centering material that is useful for light roof construction. When bent in this manner, the herringbone expanded metal is known to the manufacturers as **Trussit.** The longitudinal ribs, as at *a*, give it at least sufficient transverse resistance to allow the placing of a thin roof slab of concrete on it, without other centering or support. For this purpose, this type of slab reinforcement is excellent, the expanded metal acting both as the centering and as the steel reinforcement for the slab.

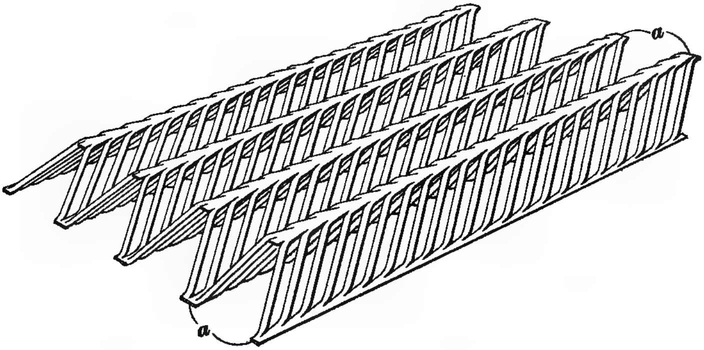


FIG. 30

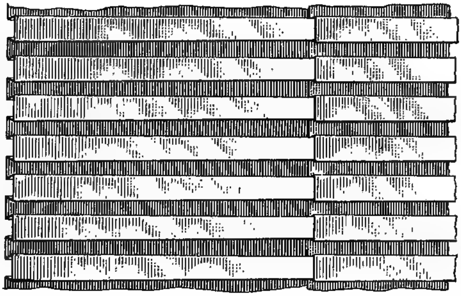


FIG. 31

43. Sheet-Metal Reinforcement. The type of metal reinforcement known as **Ferroinclave** is used to some extent for making floor slabs and for stair and roof construction. This reinforcement material consists of sheet metal that is bent into grooves, as indicated in Fig. 31, annealed sheet steel, generally of No. 24 U. S. gauge, being used in its manufacture. The corrugations are $\frac{1}{2}$ inch in depth or height and are spaced 2 inches from center to center. They are also made dovetailed in section so that the end of one sheet can slip into another, as shown. This would not be possible if the corrugations were made rectangular. The sheets are $20\frac{5}{8}$ inches in width and 10 feet in length.

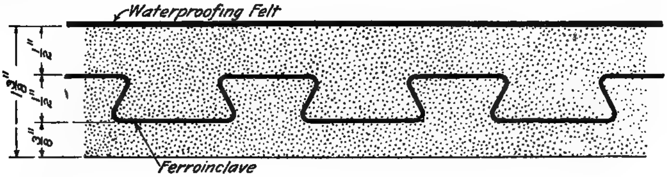


FIG. 32

Fig. 32 shows how this type of metallic reinforcement is used in constructing a slab for a light roof. The under side of the sheet metal is plastered with a mixture of 1 part of Portland cement, 2 parts of sand, and hair as required, while the covering of the upper side of the slab consists of 1 part of Portland cement and 2 parts of sand. The latter mixture is put in place before the under side is plastered.

The Ferroinclave steel plates are also convenient for constructing fireproof steps in places where it is desired to save dead-weight and to obtain a fireproof flight. This construction is suggested in Fig. 33.

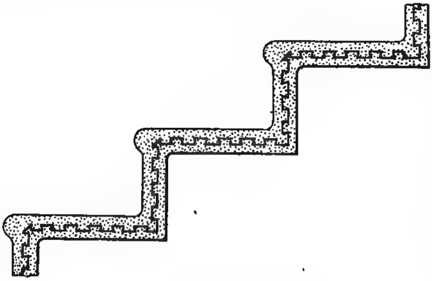


FIG. 33

44. Fabrics of Woven Wire.—There are several types of wire netting on the market that can be used in reinforced concrete construction, both for reinforcement and to prevent shrinkage. Wire netting is used for reinforcement in the construction of slabs and sometimes as a web reinforcement for beams and girders. In some instances, woven-wire fabrics have been used in conjunction with small loose rods or bars as a slab reinforcement, the loose rods or bars supplying the main portion of the metallic reinforcement and the deficit being made up by the heavy wires of the fabric. In such construction, the main rods, or bars, of the slab are sometimes wired to the fabric in order to place them accurately.

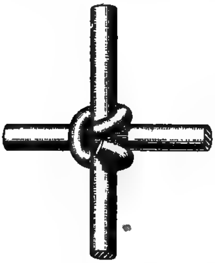


Fig. 34

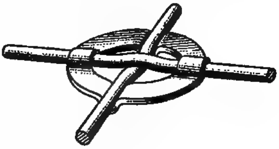


Fig. 35

Woven-wire fabrics are made by bonding, or locking, the cross-wire mechanically or by welding them electrically. All wire fabrics are constructed with transverse, or spacing, wires, and main, or carrying, wires. The *carrying wires* are the heavy wires on which the strength of the reinforcement is figured, while the *transverse wires* are those used to complete the fabric and to retain the carrying wires in their relative positions.

The wire fabrics united by means of mechanical bonds, such as the *lock-woven wire fabric* and the *tie-locked fabric*, are ordinarily arranged with the transverse wires spaced about 6 inches apart, the space between the longitudinal, or carrying, wires varying from 4 inches upwards. The electrically welded fabric is woven with meshes that are about 2½ inches square.

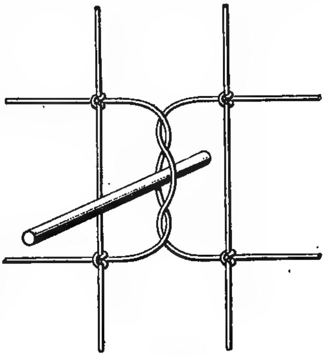


Fig. 36

All fabrics come in widths ranging from 4 to 6, 7, or 8 feet, and in lengths ranging from 100 to 500 feet, according to the weight of the wire. As the material from which these fabrics are made is drawn wire, it has a high tensile resistance and an elastic limit of from 80,000 to 125,000 pounds.

45. Lock-Woven and Tie-Locked Wire Fabric.—In Fig. 34 is illustrated the form of wire netting known as

lock-woven wire fabric. The cross-wires of this fabric are joined by means of a staple of light wire, which is bent so as to embrace the crossing wires at their intersection.

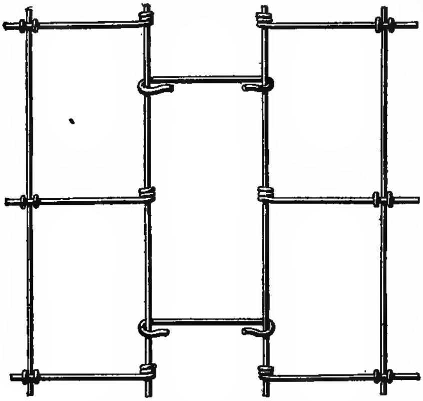


FIG. 37

Another type of junction for cross-wires is shown in Fig. 35, which illustrates the principal feature of the **tie-locked fabric.** Here the cross-wires are secured, or locked, in position by means of a small disk, or washer, and by kinking the wires.

In order that one piece of lock-woven wire fabric may be attached to another, the ends of this material are made in the form of laps. Thus, by twisting the laps of adjoining pieces in the manner illustrated in Fig. 36, an effectual connection is made. Another method of forming the junction between adjoining pieces is shown in Fig. 37. Short

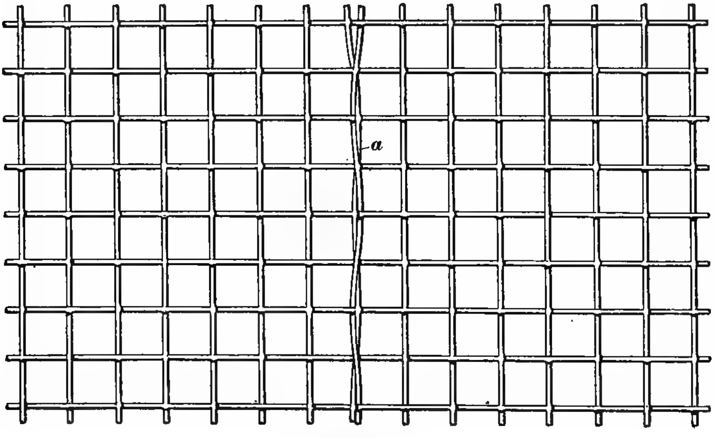


FIG. 38

wires, each having hooked ends are used. These are arranged in such a way that they form a tie and a spacer for the two pieces of wire netting.

46. **Electrically Welded Fabric.**—The **Clinton wire cloth** is a fabric that is secured at the intersections by a perfect electric weld, and it has at intervals a double wire that twines in and out, as shown at *a*, Fig. 38. The junction of the crossing wires in this fabric is very secure, as may be seen by referring to Fig. 39, which shows the perfect union of the two wires.

47. **Triangular- and Square-Mesh Wire Reinforcement.**—The two types of wire reinforcement shown in Fig. 40 are extensively manufactured for reinforced-concrete construction. The reinforcement shown in (*a*) is known as the **triangular-mesh reinforcement.** It is provided with heavy longitudinal members of either solid or stranded wire, and the cross-wires are arranged in a series of triangles that securely hold the main longitudinal wires in position. These cross-wires also provide transverse reinforcement, tending to increase the strength of the slab and to overcome shrinkage. This triangular-mesh reinforcement is regularly made in 2- and 4-inch mesh.

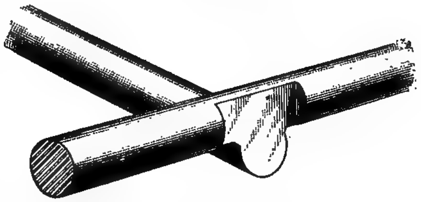


FIG. 39

48. The **square-mesh reinforcement** shown in Fig. 40 (*b*), consists of heavy longitudinal wires and cross-wires, or spacing wires. The latter are carried through and twisted around the longitudinal wires so as to form the rectangular spaces. The longitudinal tension members are made either single or stranded and are spread 4 inches apart, while the cross-wires are spaced 4, 8, or 12 inches from center to center, according to the size of the wire. This wire reinforcement is made in standard rolls of 150-, 350-, and 600-foot

lengths. In using reinforcement of this kind, the widths may be lapped, the distance between the longitudinal wires forming strips of additional reinforcement through the slab

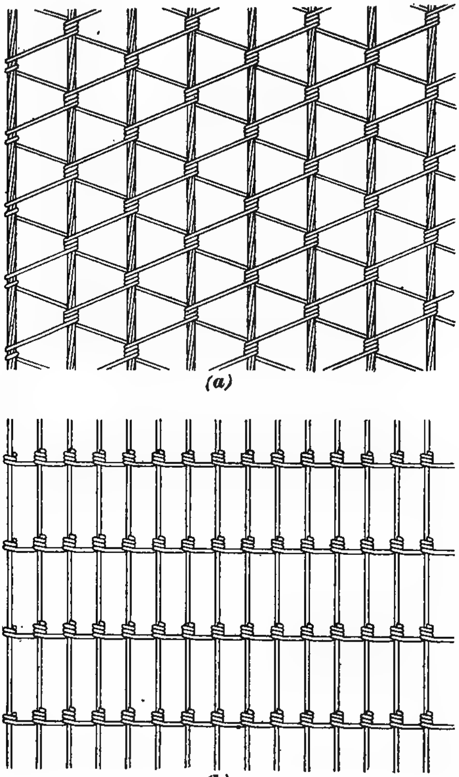


Fig. 40

construction. The material may also be wrapped around the vertical rods of the column reinforcement so as to form a tie for it.

SYSTEMS OF STEEL REINFORCEMENT

SELECTION AND CLASSIFICATION

INTRODUCTION

1. By observing the actual work performed in erecting reinforced-concrete structures, it is usually evident that a great deal depends on the workmanship displayed in constructing the forms, in locating the steel, and in mixing and placing the concrete. If all these processes are properly carried out and the work is well designed, it can be safely said that, from the point of strength and durability, there is little difference between the various systems of reinforcement and that the advantage of one over another exists only in the economy of material and labor. Most systems include beam-and-girder construction, and sometimes combine as well a particular type of column reinforcement.

In conjunction with the floor systems and column supports, there are many different forms of wall construction and many peculiarities that are individual to reinforced concrete, and to be successful the designer of reinforced-concrete structures must be well informed about them. These matters, however, will be treated in another Section. It is the purpose of this Section to describe only those systems of reinforced-concrete construction which have proved to be practical and which are being successfully used more or less throughout the country in the erection of reinforced-concrete structures.

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GENERAL CLASSIFICATION

2. While it is difficult to classify the many kinds of reinforced-concrete construction, they can be divided, according to the nature of the steel reinforcement, into two primary types, namely, the *loose-rod systems*, and the *girder-frame*, or *fabricated, systems*. All other systems are here classed as *miscellaneous systems*.

In the loose-rod systems of construction, the reinforcing rods, or bars, are delivered at the building as they come from the mill, being bent to the required form on the ground and embedded in the concrete one by one as the work progresses. In the girder-frame, or fabricated, systems of construction, the several rods, or bars, that form the reinforcement of the beam or girder are wired, secured with clips, or otherwise fastened together; that is, they are fabricated into a complete reinforcing element for each structural member. The assembled frames thus fabricated are set in the forms and secured therein before the concrete is placed, or poured.

3. Before selecting either of these two general types of reinforced-concrete construction the several advantages and disadvantages of each should be carefully considered. If it is possible during the operation of construction to have a competent and experienced superintendent constantly in charge of the work, the loose-rod system of reinforcement can be used with entire safety, for by the exercise of vigilance on his part, the reinforcing rods belonging to each beam or girder will be placed according to the requirements of the drawings, and excellent construction will result. If, on the other hand, the conditions under which the construction is undertaken are such that it is impossible for the work to receive the careful attention of an experienced superintendent, then there will be considerable advantage in employing the fabricated system of steel reinforcement.

Experience teaches that with careful workmanship the loose-rod system of reinforcement is as efficient as the one embodying the use of a girder frame, or fabrication of rolled rods. However, the latter system eliminates to a great extent the uncertainty attendant on careless workmanship or oversight on the part of the superintendent.

In general, the loose-rod system of construction is not so expensive as the fabricated system, because the latter system requires considerable more shop work, while the loose-rod system requires only a little more labor in erection. Besides, the typical loose-rod system is unprotected by patents, so that no additional charges have to be made to cover royalties.

4. In addition to the systems of reinforced concrete just considered, there are important systems of steel reinforcement that are based on the use of rolled-steel sections. The columns, beams, and girders used in these systems are reinforced with a light structural framework that is rigidly riveted, or bolted, together at the connections, and is of sufficient strength to support the dead load of the wet concrete. Such systems may be classified as *steel-frame systems*.

In other systems, classed as *lintel*, or *self-centering*, *systems*, the wooden centering ordinarily used to support the concrete during construction is eliminated. The blocks, slabs, or lintels used are molded separately and are so devised that they can be put in place after they have set.

Another interesting system—the *composite system*—consists of a combination of slow-burning construction and reinforced concrete.

REINFORCEMENT SYSTEMS

LOOSE-ROD SYSTEMS

5. Typical Loose-Rod Systems.—A complete floor system constructed of **loose rods** is shown in Fig. 1. This figure illustrates the girder, beam, slab, and column reinforcement, as well as the auxiliary features of the construction, such as the shrinkage rods in the slab, the stirrups, the sockets, and the pipes for the passage of electric-light wires or other installation. In this instance, the entire floor system and column supports of the building are of reinforced concrete, while the walls may be either of brick or of reinforced concrete.

The beam reinforcement consists of three reinforcing rods. Two of these rods run straight through the entire length of the beam, as at *a*, while the third one is bent upwards at the ends, as at *b*. This bent member provides tensile resistance at the top of the beams and thus takes care of the negative bending moment, which occurs in all beams fixed at the end. The bend in such rods is usually made at an angle of about 30° with the horizontal, and in making it care should be taken to see that the rods are straight at the center of the span for at least one-third the distance between the supports.

Where two beams join a girder at opposite sides, the rods of the beam, especially the bent rod, as at *b*, should lap the rods of the opposite abutting beam several feet. A tie-rod *c* that is 4 or 5 feet in length, and sometimes bent down at the ends, should be placed over the top of the beam juncture. If two girders abut on opposite sides of a column, the tie-rods shown at *f* are used.

The girder reinforcement consists of five rods, two of them being bent up, as shown at *e*, to provide against negative bending moment. Frequently, two beams bear on each side of a girder between columns, instead of one at the center of

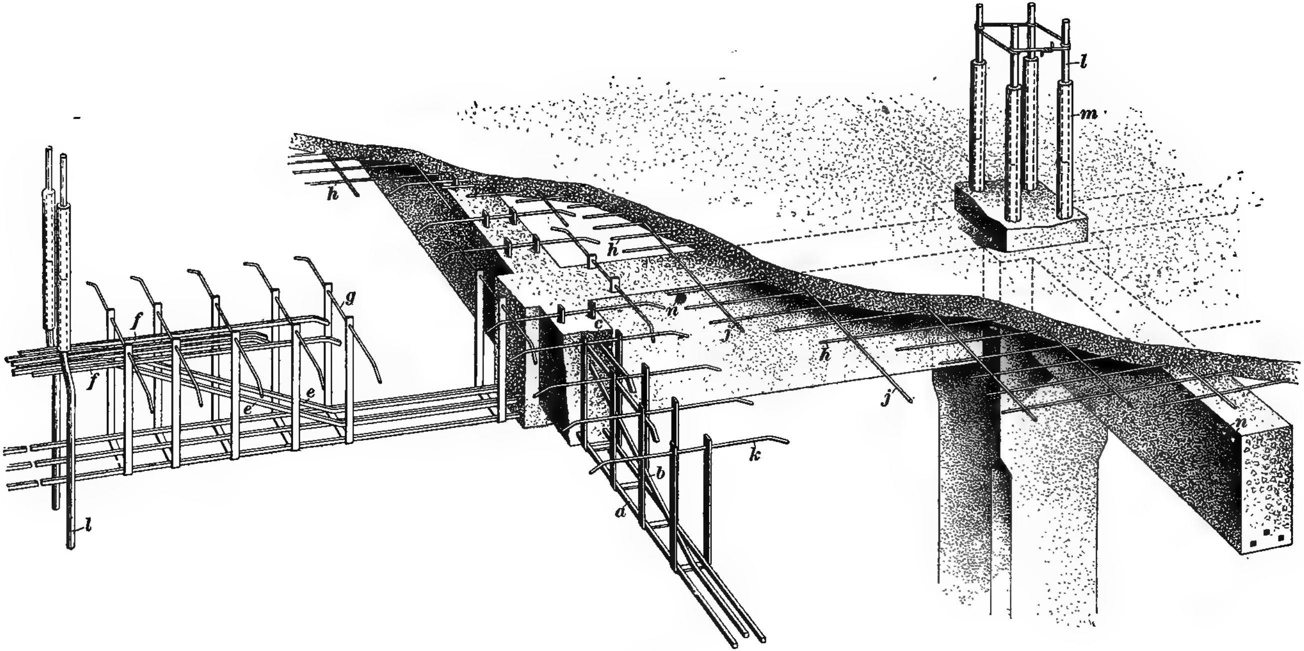


Fig. 1

the girder span, as shown in the illustration. In such cases, it is customary to shorten the length of the bend in the rods *e*. In this way the length of the straight part is increased and the bend is not made too near the juncture of the beams and girders; thus, the strength of the girder is not decreased at the points of greatest bending moment. In good practice, the bottom rods of the girder, or those which run through the entire length, continue as far as the center of the columns, while the top rods overlap a distance of from 6 inches to several feet.

In the best work, two short rods *f*, already mentioned, are located transversely through the column. These rods tie the adjoining girders together and provide additional rigidity at the junction of the girders with the column. They also insure the required strength of the beams or girders fixed at the ends. These lap rods may be left straight or they may be bent down at the ends. However, it is usually considered unnecessary to bend these rods at the ends, as the bond between the steel and the concrete, provided the rods are 4 feet or 5 feet in length, is sufficient to realize their full strength.

6. The construction illustrated in Fig. 1 shows that neither the beams nor the girders are provided with brackets at their junctions with the columns. In buildings of ordinary height, where the beams and girders are of good depth, these angle brackets are not necessary. However, they always insure greater strength at the junction of the beams with the columns, and, besides, they tend to strengthen and stiffen the building to a great extent, as well as to decrease the clear span of the beams and girders. Such brackets are generally reinforced with about two ½- or ⅝-inch rods. These rods must lap the girder and column reinforcement, it being well to wire them to the metal in the columns and girders in order to fix them in position.

7. The slab rods, or the reinforcement of the concrete slab that spans the distance between the several beams, are illustrated at *h*. These rods are generally spaced at about

6 inches from center to center and are from $\frac{5}{16}$ to $\frac{5}{8}$ inch in diameter. Sometimes, however, square, twisted rods are used. In such cases, the rods are of the same size and are spaced in the same way. The slab rods should bond with the stirrups, or web reinforcement, of the beams, and may be either threaded through, interlocked, or wired to them. It is customary in the typical loose-rod construction illustrated to provide shrinkage rods that extend at right angles to the regular slab reinforcement, in order to prevent shrinkage cracks in the concrete. For this purpose, $\frac{1}{4}$-inch round or square rods *j* are generally used, and these are spaced about 2 feet from center to center. In order to bond the concrete over the main girders securely, it is also good practice to provide over these important members rods of about the same size as the slab rods. These rods should run through holes punched in the top of the stirrup, as illustrated at *g*, and should extend at right angles to the axis of the girder. Such rods tend to bind and tie together the concrete of the slab directly over the girder, thus adding to its compressive resistance and insuring its unity of action with the concrete of the girder. Sometimes, similar rods are used in the slab over beams, as shown at *k*. These rods also help to resist the negative bending moment in the slab.

8. The longitudinal reinforcement of the concrete columns consists of four round rods *l*. Rods long enough to extend through several stories can be obtained, but it is customary to project them above the concrete of each story about a foot and to splice them by lapping and wiring or by using pipe sockets *m*, as illustrated. Pipe of standard size is used for the sockets, and they are made of such length that when they rest at one end on the finished concrete work they project far enough above the end of the rods to provide a socket for those of the upper stories.

9. In the construction of high buildings, it is good practice to provide additional strength at the junction of the longitudinal reinforcing column rods with lap rods. These lap, or splice, rods are usually from 3 feet to 6 feet in length, one-half of their length projecting into the lower column, or finished concrete work, of the story below.

In the construction under discussion, the several longitudinal rods forming the reinforcement for the concrete columns are tied together at intervals of 12 inches with ¼-inch, round-iron loop ties, as shown. These ties are arranged as the concrete is poured and tamped in the forms. For ordinary construction in which the columns are subjected to a pressure not greater than 500 pounds to the square inch, these loop ties may be of the size just mentioned, and they may be spaced at a distance apart not greater than the least width, or diameter, of the column.

10. In the construction of reinforced-concrete warehouses, factories, or buildings of a similar nature, bolt sockets should be embedded in the soffit of all beams and girders. These sockets should be placed in a row on every beam and girder, and at distances of about 4 feet from center to center, the first and last socket in each row being placed, preferably, near a column. Such sockets are convenient for fastening wooden sleepers for the support of shafting, or for securing batten boards to cover electric-light wires. They may also be used to support sprinkler or steam pipes or overhead trolley tracks for hoists. Such sockets should be tapped for ¾- or ⅞-inch bolts, or they should be arranged to receive **T**-headed bolts. They may be constructed in numerous ways, as described in a subsequent Section.

11. The cutting of reinforced concrete should be avoided, not only on account of the labor involved, but also because the structure is liable to be weakened or, at least, its appearance disfigured. Forms, or holes, required for installations should, therefore, be molded in the concrete work during progress of erection. Frequently, it is not possible to lay out beforehand the electric-light or power wiring, but if this installation is to be adopted, 1¼-inch pipes should be embedded near the center of the span of all beams and girders, close to the under side of the slab construction, as at *n*. These pipes will provide a passageway through such parts of the con-

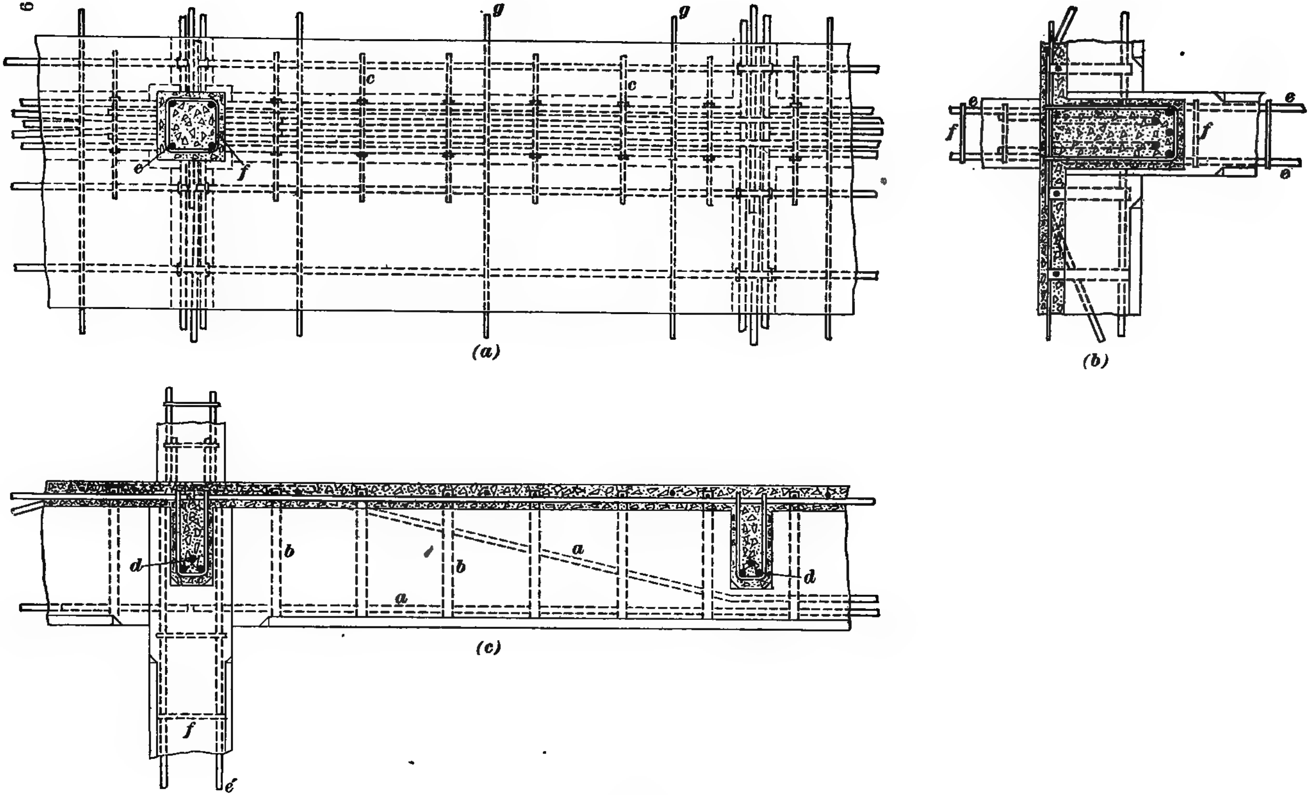


FIG. 2

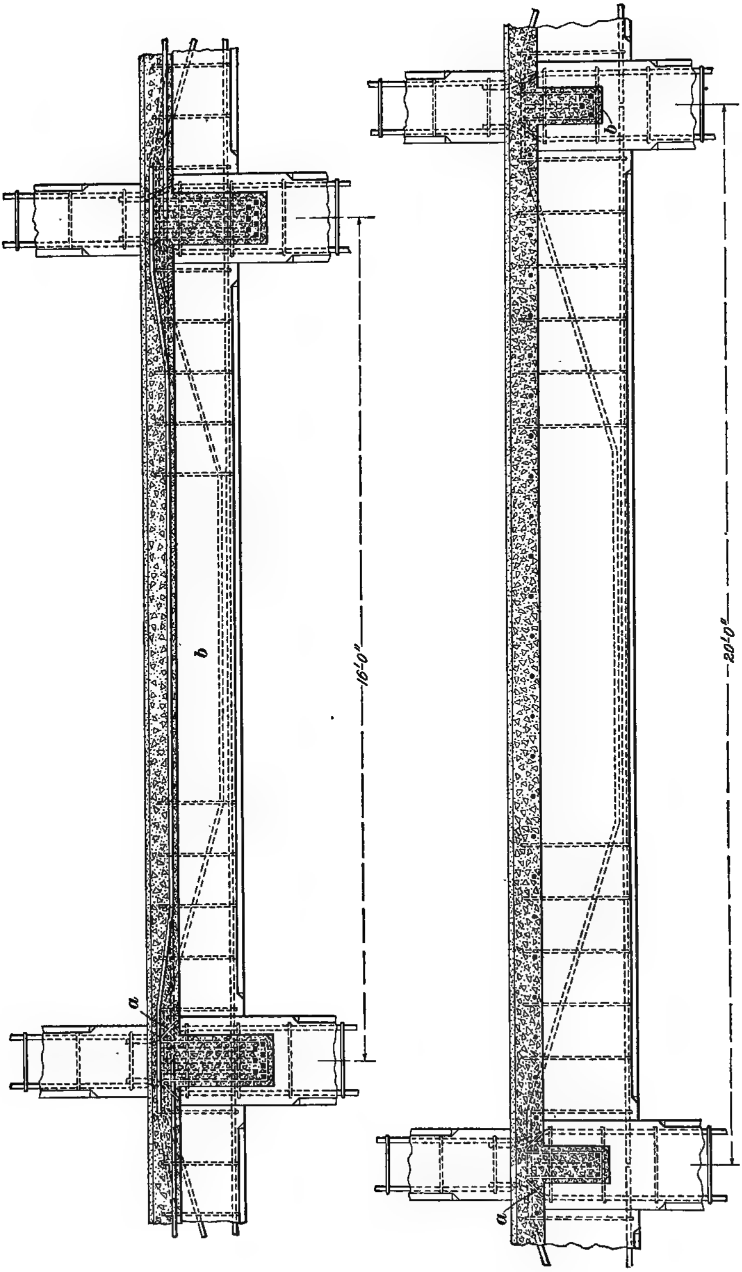


FIG. 3

struction for the wires or electric conduits. Pipes can be furnished and embedded in the concrete work at a small cost, and when placed as illustrated in the figure, any lighting or power wires can be run close to the slab and need not be carried down the cheeks and under the soffits of beams and girders, unduly exposing them and making a clumsy installation.

12. Interesting examples of well-designed, typical, reinforced-concrete construction are illustrated in Figs. 2 and 3. Both of these illustrations show designs of the American Concrete Steel Company.

Fig. 2 illustrates a plan (*a*) and two sectional views (*b*) and (*c*) of the system of construction used for short spans. In view (*c*), 1⅛-inch steel rods are used for the main reinforcing bars *a*. The bottom bars of the reinforcement extend to the center line of the column bearing, while the truss rods lap the column bearing from 1½ to 2 feet. The stirrups *b*, which pass under the reinforcing rods, consist of ⅛″×1¼″ bar iron bent in **U** form. When located in a beam, these stirrups are punched at the top so that some of the slab rods can pass through them, as shown in (*c*). In the case of a girder, the short shear rods *c* pass through them, as shown in the plan view (*a*). The beams are reinforced with three rods *d*, and the web stresses are taken care of by stirrups similar to those used in the girder. The columns are reinforced with longitudinal rods *e*, which are tied at intervals with ¼-inch steel binders *f*, the latter being bent around the rods and wired to them. Through the floor slab there is also placed at intervals ¼-inch steel shrinkage rods *g*, as shown in the plan.

Fig. 3 illustrates the construction used for long spans. There are no intermediate beams in this construction, and the floor slabs are designed for clear spans of from 16 to 20 feet. Such a construction as this can be used with light floor loads, especially if the slab is reinforced in both directions. In the design shown, the stirrups, instead of consisting of ⅛″×1¼″ bar iron, as in the preceding construction, are made of ¼- or ⅜-inch round bar. These stirrups, which are bent up in

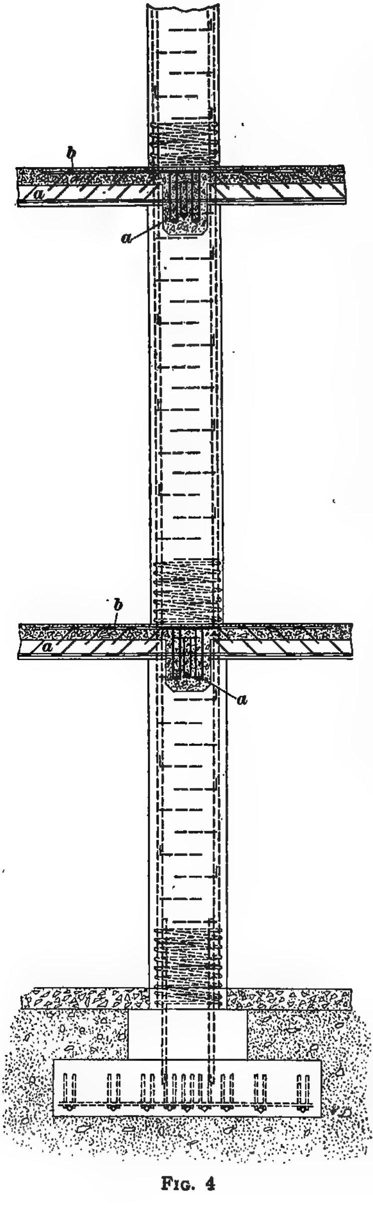


FIG. 4

U form, pass under the reinforcing rods, and their ends are bent in the form of a hook, as shown at *a*. By means of this hook the stirrups can be arranged to support the reinforcing rods from the centering. It is good practice to arrange some of the slab reinforcement in such a way as to interlace with the stirrups. It will be noted from the figure that alternate rods of the slab construction are brought toward the top of the slab, over the girder and beam bearing. This is for the purpose of resisting the reverse bending movement. If the slab is not reinforced in both directions, the beams *b* act merely as ties, or stiffeners, for the construction, as the slab spans in all instances from girder to girder. If the slab is reinforced in both directions, the beams will assist in supporting the load, and they should therefore be reinforced for this purpose.

13. Kahn System. The **Kahn system** of reinforced-concrete construction is based on the use of the trussed bar described in *Elements of Steel Reinforcement*. In this system, this trussed bar is used as the reinforcing member of the beams, girders, and columns used in factory or warehouse construction; or, it may be used with concrete in conjunction with hollow terra-cotta tile for floors that are designed with a minimum dead load and generally employed for a light live load. The latter construction is suitable for hotels, office buildings, places of public assembly, and fireproof dwellings.

14. A typical monolithic construction based on the use of the Kahn system is shown in Fig. 4, which is an elevation, partly in section. In this figure, the main reinforcing members of the beams and girders are shown at *a*, and they consist of Kahn bars with the prongs bent upwards so as to form stirrups. The bars are usually placed in the forms on a 2-inch bed of concrete, and after being centered, or registered in their proper positions, they are secured by wiring or by blocking. In order that the girders and beam connections over columns may have continuity, an inverted Kahn bar *b* is used over such junctions. This arrangement furnishes at the top of the beam material that will resist the negative bending moment. This feature of the construction becomes very efficient when the depth of the beam or girder is such that the prongs of the inverted bar interlace with those of the main reinforcing trussed bar.

In the construction of columns, Kahn bars are used as longitudinal bars, to prevent failure by lateral flexture. As a rule, four Kahn bars are employed. These are spaced in each corner of the column, as shown in Fig. 5, with the prongs bent at right angles. Sometimes, junctions between the reinforcing rods of the columns are made by means of pipe sleeves, or else a clamping device is used. In all instances, splice, or lap, bars should be used in the construction of important columns, so as to add greater security at the connections. The bars are usually tied together with heavy wire or light rods, wound around them as shown part way up the columns in Fig. 4.

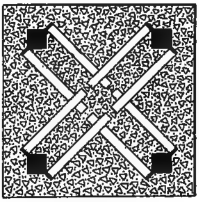


Fig. 5

15. Fig. 6 shows a girder frame of a special construction that has been evolved by fabricating the Kahn bar into a frame that is similar to the unit frame, which will be described later. Owing to the shape of the bars, a specially shaped clamp *a* is employed. It will be observed that the stirrups form an efficient means of keying and binding in the concrete, and by using the bars as a girder frame, any chance of misplacing the steel, no matter how remote, is obviated and the position of the bars in the forms is insured.

In the illustration, the straight reinforcing bars are shown at *e*, and those bent up to counteract the negative bending moment are shown at *d*. The two sets of bars are held together by clamps *a*, the connecting bolt having a socket *b* at its lower end for the purpose of facilitating the fastening of sleepers, etc. The rods are held in the desired positions by means of the spacers *c*. Another device for holding sockets in place is shown at *f*.

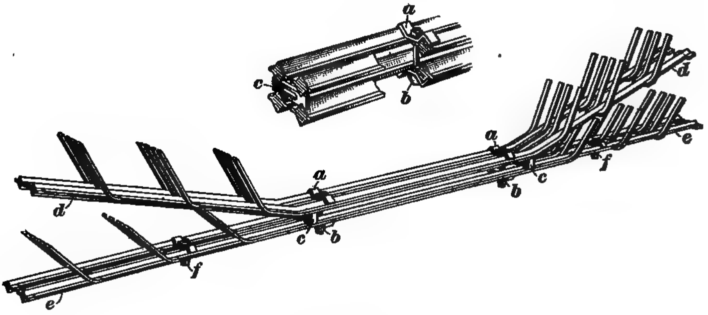


FIG. 6

16. As previously mentioned, the Kahn bar is frequently used with concrete and hollow terra-cotta tile to form floor systems designed for light loads. This type of construction is illustrated in Fig. 7. In view (*a*) is shown a cross-section of the construction through the beams, or secondary members, of the floor system. It will be observed that the hollow tile is spaced sufficiently far apart to permit a narrow reinforced-concrete joist, or beam, to be formed between them. Usually, the tiles are from 12 to 16 inches in width and from 6 to 12

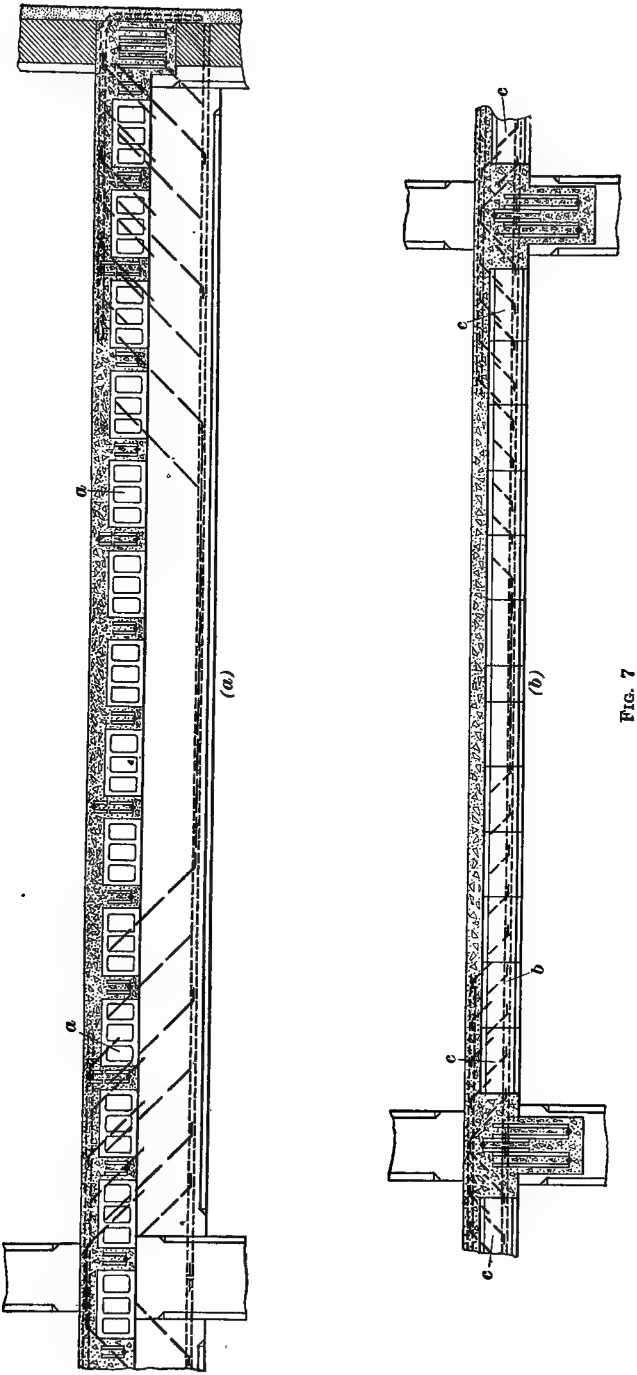


FIG. 7

inches in depth, the hollow part having the cross-section indicated at *a*. A centering on which the tiles are laid is used in this system of construction. The tiles are spaced about 5¾ inches apart and in the space between them is laid concrete that is reinforced with Kahn bars. The concrete is carried over the top of the tile to a depth of about 2 inches. The Kahn bar shown at *b* in (*b*) is provided with the usual prongs, or stirrups, which are bent upwards toward the abutments. The tiles are serrated, or corrugated, along the sides, so as to provide some shearing strength in addition to the adhesion of the concrete. By running the concrete over the top of the tiles, the advantage of a **T**-section is obtained for the beams or joists.

For economy and for strength, it is often necessary to make the main girders of a reinforced-concrete building of **T** section. These girders in the Kahn hollow-tile system support the ends of the concrete joists that run between the tiles. Therefore, to effect this **T** section, the tiles are stopped off one or two rows from the girder on each side. A section through this construction is shown in Fig. 7 (*b*). As will be observed, the tiles *c* stop a considerable distance from the center of the girder and the space between the ends of the tiles is filled with concrete incorporated with the concrete forming the girder. In this way, the desired **T** section and, consequently, the necessary resistance to compression at the upper portion of the member are obtained.

In very light construction, the tiles are frequently extended over the top of the girder and the girder is made amply wide so as to provide resistance to compression in the upper portion; or the girder is doubly reinforced by putting bars, or rods, in the top of the section as well as in the bottom.

17. During the construction of this floor system, the tiles should be held temporarily in position either by placing blocking between them or by nailing them to the forms. Thus, when the concrete is tamped in the intervening spaces, the tiles will not be misplaced. Also, in constructing the thin slab over the top of the tile, it is good practice to use small aggregates. The strength of this portion of the work would be of little value if stone of large size were used or if the mixture were lean.

It is not customary in this type of construction to use shrinkage rods in the slab that is laid over the top of the tile, though there is no reason why such rods cannot be used to advantage. Since the tiles are quickly laid on the centering and, consequently, minimize the quantity of concrete required for the construction of the floor, and, also, since the concrete placed in conjunction with the tiles sets more rapidly than when placed alone, this particular type of construction can be carried on with considerable rapidity. Such a floor construction as this, embodying tile having a hollow air space, acts as a good deadener of sound and also tends to prevent heat from passing from one floor to another.

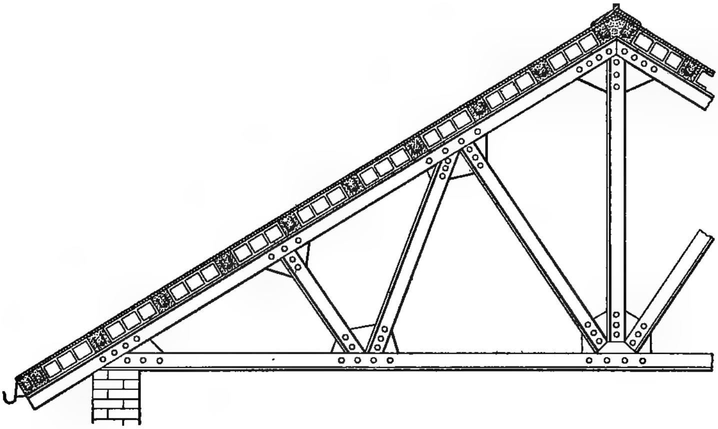


Fig. 8

18. In Fig. 8 is shown a typical example of a roof construction in which the Kahn system just explained is employed. As will be observed, the hollow tiles are very wide and shallow, and thus increase the distance between the concrete joists. The use of wide tile, however, is permissible only where light loads are expected.

19. Merrick System.—In Fig. 9 is shown the **Merrick floor system.** This system is intended more for fireproof floor construction than for the construction of an entire reinforced-concrete building; nevertheless, it adapts itself to the latter use with any style of concrete girders and columns. The space between girders is occupied by narrow concrete beams *a*, as shown in the transverse section (*a*). Between these beams are placed boxes made of one of the many styles of metal fabric on the market. These boxes run the entire length of the clear span of the floorbeams, as shown in the longitudinal section (*b*), and serve to make the floor lighter. They are indicated in both views by the heavy dotted lines *b*.

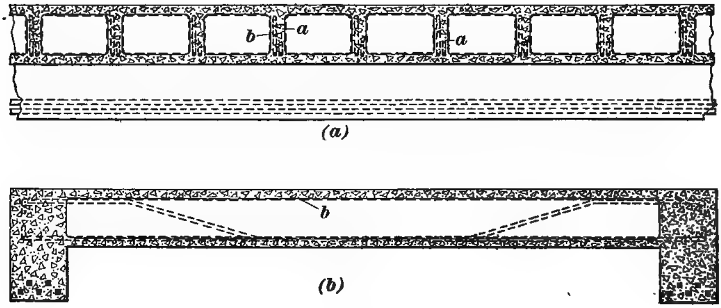


FIG. 9

Above and below the metal-fabric boxes is a layer of concrete. This layer is usually made about 2 inches thick, so as to give a flat ceiling and a flat floor surface.

In laying this floor, a wooden centering, with a flat surface, is formed, and on it is laid 2 inches of concrete. The fabric boxes and metal reinforcement are then placed, and the space between the metal boxes is filled and well rammed with concrete. The top layer of concrete is then placed directly on the metal boxes.

20. Hennebique System.—The **Hennebique system** of construction has been used to a great extent in the United States, and is similar to the loose-bar system previously explained. The Hennebique system takes its name from its inventor and licensor, and consists primarily of the special constructive features shown in Fig. 10. As shown in the illustration, a reinforced-concrete system of girders and beams supports a floor slab of the same material. The reinforcement of the beam consists of two round rods. The bottom rod *a* extends straight throughout the length of the beam, while the top rod *b* is bent upwards at an angle. In order that these rods may have a greater grip, or bond, in the con-

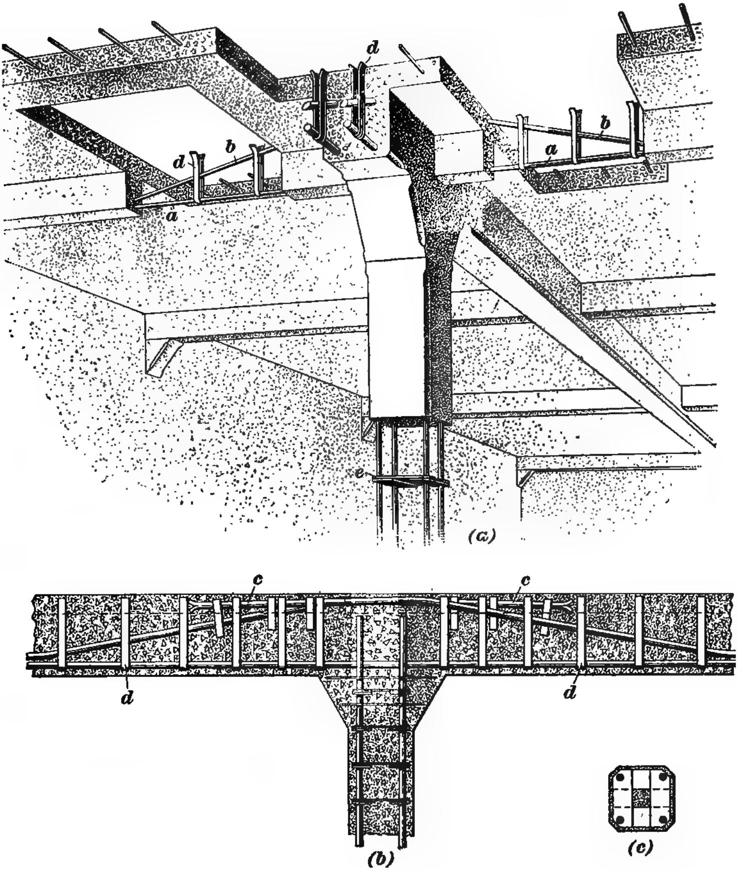


FIG. 10

crete they are split and swaged out at the ends *c*, as shown in view (*b*). When necessary, more than two rods may be used, as shown in the foreground of view (*a*).

In this type of reinforcement, the web stresses, namely, the vertical and oblique shearing and tensile stresses, are resisted by stirrup reinforcement made up of **U**-shaped pieces *d* of bar iron that is about $\frac{7}{8}$ inch wide and $\frac{3}{32}$ inch thick. These stirrups serve to hold the upper rods *b* in position, and are of such length as to extend into the slab concrete. They thus furnish the necessary resistance to transmit the stresses from the beam to the slab. As will be observed from the illustration, the top ends of the stirrups are bent over so as to give additional bond in the slab.

In the slab construction of this system, usually every other rod is bent up over the beam bearings. Where the slabs are thick, short stirrups are frequently provided in conjunction with the slab reinforcement. These stirrups are of the same type as those used in the construction of the beam. They are spaced close near the bearings of the slab, but sometimes they are omitted entirely as the center of the span is approached.

The Hennebique system of reinforced-concrete construction is employed either for slab, beam, and girder construction, as described, or it may be used for thick slab construction that is supported on four sides by walls or girders. In the latter case, the term *Hennebique flat-plate system* is applied to the construction. In this type of the system, double reinforcement is sometimes used, the rods extending both ways and being alternately straight and bent.

The columns are reinforced with longitudinal rods or bars that are tied together and held secure by plate or bar-iron ties *e*, as shown in view (*a*), or in detail in view (*c*).

The principal and distinguishing features of the Hennebique system are: (1) the use of a straight and bent rod, with ends split and swaged out for additional bond; (2) the use of the flat bar-iron stirrups bent in **U** form and turned out at the ends; (3) the use of the alternating straight and bent rods in the slab construction; and (4) the extension of the bent-up

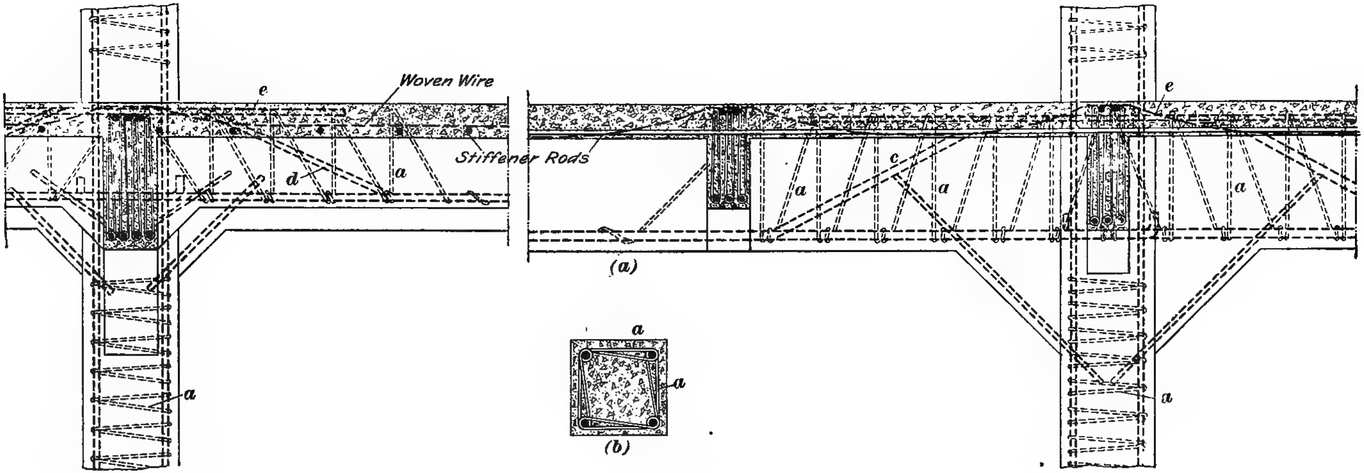


FIG. 11

or truss rods over the supports which acts as a tie and resists the negative moments.

21. Gabriel System.—The **Gabriel system** of reinforced-concrete construction consists of steel reinforcing bars to which are attached round-iron stirrups. These stirrups are formed by wrapping a wire *a* several times around the reinforcing bar and extending it up into the slab, as illustrated in Fig. 11, which shows this particular type of construction complete and in its several details. The wiring of the columns is continuous, and extends from the bottom to the top, as indicated in the figure, particularly in the sectional view (*b*). Several of the reinforcing rods of both the girders and the beams bend upwards, as shown at *c* and *d*, respectively They also lap over the center of the column to form

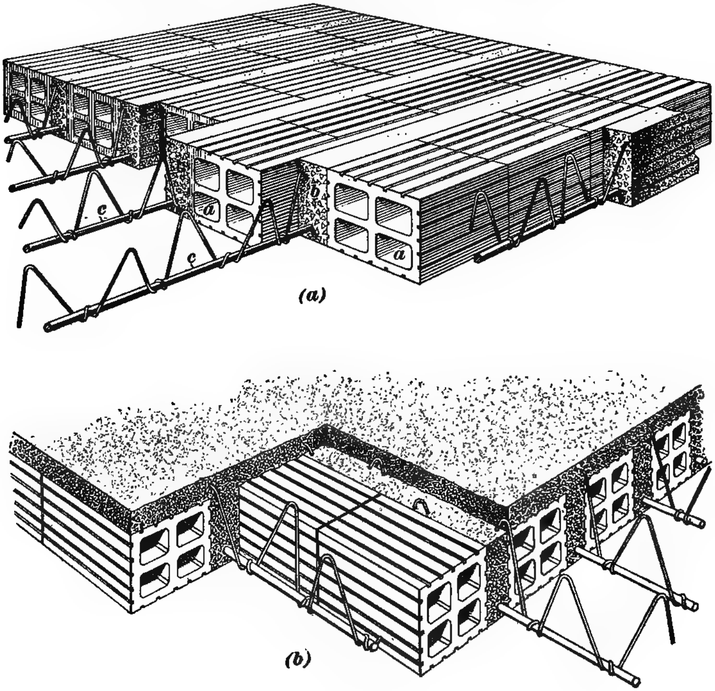


Fig. 12

an additional bond in the concrete, as shown at *e*. In this system of construction a lap bar is sometimes provided at the top of the junction as shown. With this system any method of slab reinforcement can be used for ordinary construction, and in Fig. 11 the slab is shown reinforced with woven wire and shrinkage rods of round iron.

With the Gabriel system of floor construction, hollow tile is also used in a manner similar to that described in connection with the Kahn system. These combination reinforced-concrete and hollow-tile floors are illustrated in Fig. 12. The system shown in view (*a*) is suitable only for very light construction. The hollow tiles *a* are sandwiched between reinforced-concrete joists, as at *b*, and the reinforcement of the floor consists of a main reinforcing bar *c*, around which a round-iron stirrup is wrapped so as to extend continuously. A similar construction is shown in view (*b*). Here, however, a slab of concrete is extended over the top of the tile, thus giving additional compressive strength at the top of the concrete beams and permitting this construction to be used for heavier loads. This type of slab construction is suitable for moderate loads.

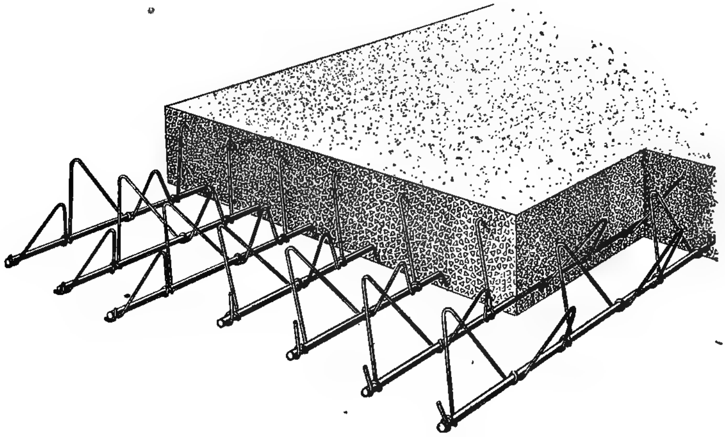


FIG. 13

In Fig. 13 is shown the method of reinforcing a heavy floor slab. In this case, small reinforcing bars and continuous stirrups typical of this system of construction are used.

The other features shown in Fig. 11 are typical of reinforced-concrete, loose-rod systems, and do not really belong to the Gabriel system. The peculiar method of wrapping and extending the light bar-iron or wire stirrups is the essential feature of this construction.

22. DeVallière System.—The elementary principles of the **DeVallière construction** are illustrated in Fig. 14. It consists of a single, straight reinforcing bar and a continuous wire stirrup. This wire is looped around the bar and extends up to nearly the top surface of the slab or beam, as shown at (*a*). If more than one rod is used for the reinforcement of the beam or girder, an effort is usually made to stagger the wire loops, or stirrups, as indicated in view (*b*). Such an arrangement of stirrups furnishes an excellent metallic bond throughout the body of the beam and also a means by which the slab rods may interlace with the web reinforcement.

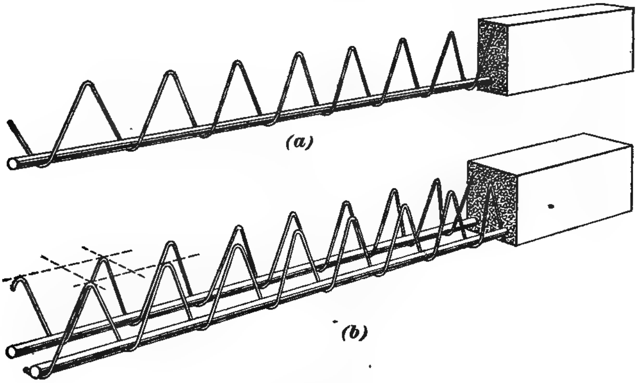


FIG. 14

The DeVallière system has been modified considerably by the adoption of the system of construction shown in Fig. 15. This modified system has been used extensively in Italy, Switzerland, and somewhat in the United States. A very similar construction is being used in conjunction with most of the loose-rod types of reinforcement.

The particular features of the construction shown in Fig. 15 are the plain, round, straight-rod reinforcement, combined with the bent, or truss, rod, and the heavy, round, twisted stirrups. These stirrups engage the bottom rods of the reinforcement, and are twisted so as to form a loop at the upper end for the passage of the rods that form the slab reinforcement. In this manner, the elastic limit of the stirrup is increased by twisting, and there is a positive bond between the reinforcing rods of the girders and the reinforcement of the slab. In spacing the wire stirrups, they should be so arranged that their distance apart from center to center of loops is in multiples of the rod spacing in the slab, so that they will be in position to receive some of these rods.

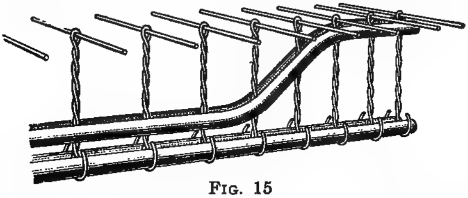


FIG. 15

23. American System of Reinforcing.—The system of reinforced-concrete construction shown in Fig. 16 is known as the **American system.** It is based primarily on the use of rods or bars for the main reinforcement of columns, beams, and girders, combined with the use of lock-woven wire fabric for the reinforcement of slabs and for the web reinforcement of beams and girders. The wire fabric is also used to hold the main reinforcing bars of the columns in place, the wires of the fabric acting as ties.

In the illustration, the main reinforcing bars of the longitudinal beams are shown at *a* and the wire fabric used for the stirrups, or the web reinforcement of the beam, at *b*. This wire fabric is bent to the shape of the beam and passes under the reinforcing rods, and is turned at the ends at right angles, parallel with the slab for a distance of several meshes. The beam is reinforced with rods conforming to the same shape,

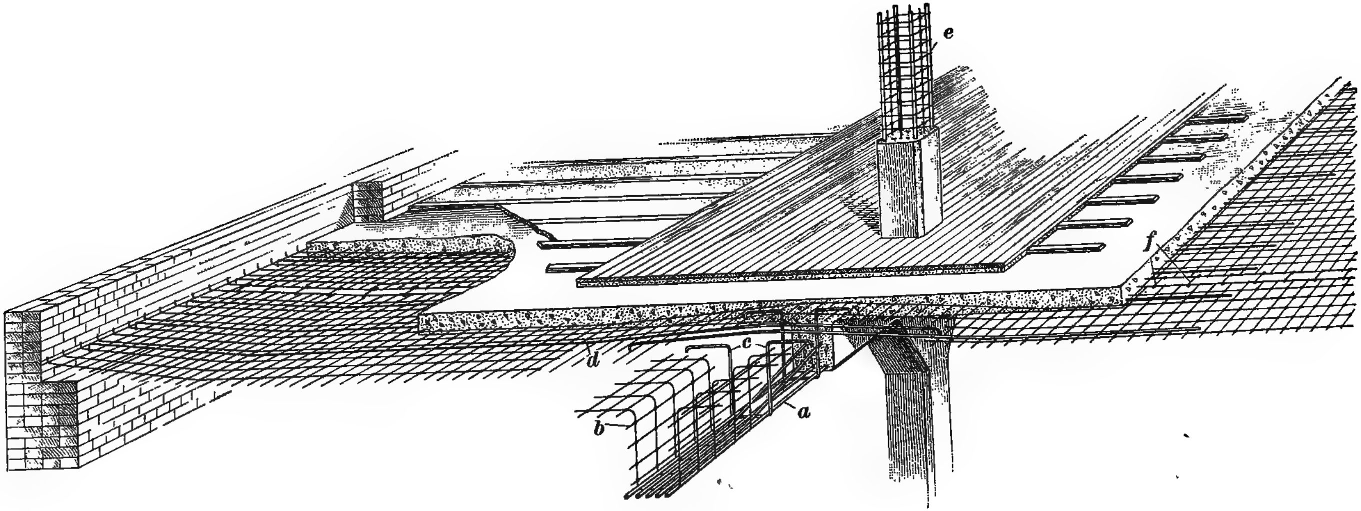


FIG. 16

as shown at *c*. If more reinforcement than that furnished by the wire fabric is required for the slab construction, additional rods, as shown at *d*, may be used, and in order to space them properly, they may be held by wiring them to the fabric. In this construction, the strips of wire fabric forming the slab reinforcement have their sides joined with hooked fastenings *f*. This device saves the material from being wasted where a lap is made; also when such a junction is bedded in the concrete work, the hooks provide ample resistance against shrinkage. These hooks are of a length that insures the proper spacing of the main reinforcing wires.

The typical reinforcement of a concrete column is shown at *e*. This column is reinforced with four longitudinal rods, the netting being wrapped around these rods and tied at intervals with wire.

The principal advantages of a reinforced-concrete system of this type are that the reinforcement of the slab is properly distributed, and that ample provision is made against shrinkage in the concrete by the cross-wires of the wire fabric. On account of the high tensile strength of the drawn wire of which the woven-wire cloth is formed, considerable strength is gained in this type of construction. This wire generally has twice the tensile strength of ordinary rolled steel bars. Another advantage consists in the use of small units and their equal distribution throughout the concrete work in which they are embedded. The use of a fabric for the reinforcement of floor slabs facilitates the operation in the field, because the fabric is shipped in rolls of from 4 feet to 6 feet in width and in lengths of several hundred feet. This size of roll makes it possible to place the wire reinforcement in a continuous strip from one end of the building to the other, absolutely insuring continuity in the floor-slab construction.

24. Mushroom System of Reinforcement.—A system of reinforced-concrete construction that differs from all others in that no beams nor girders are used throughout is illustrated in Figs. 17 and 18. This system of construction, termed the *mushroom system*, consists of reinforced columns

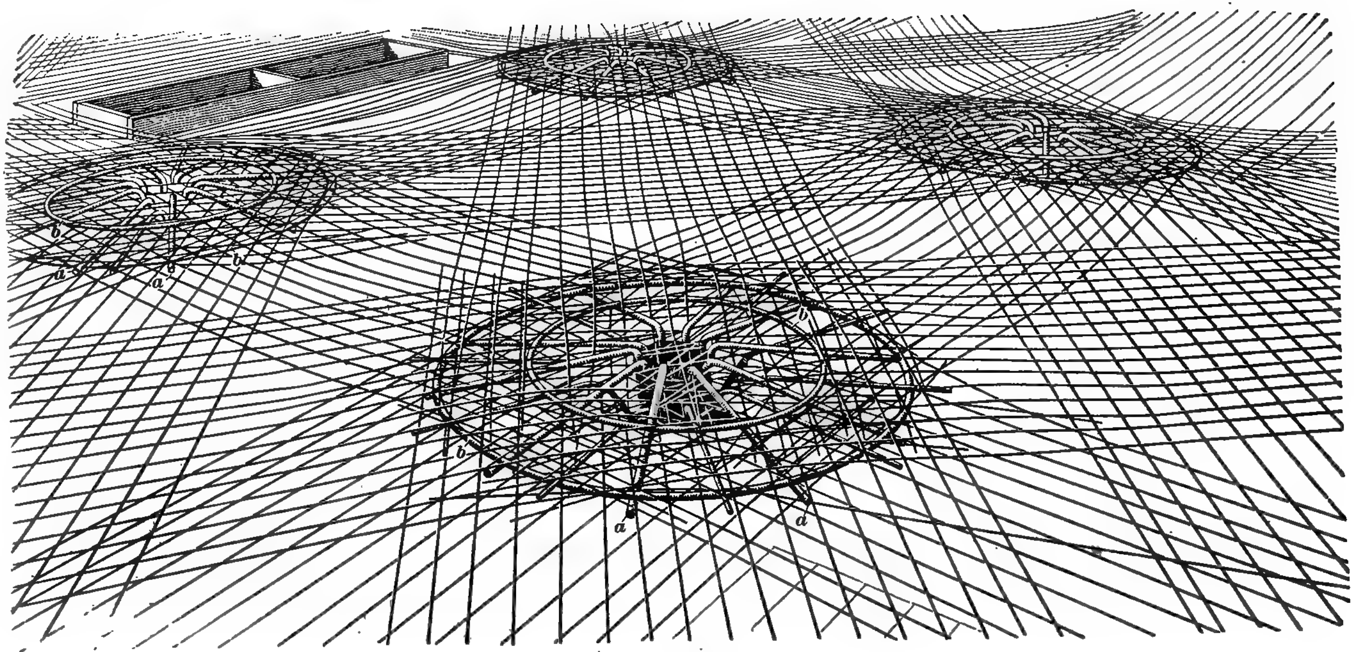


FIG. 17

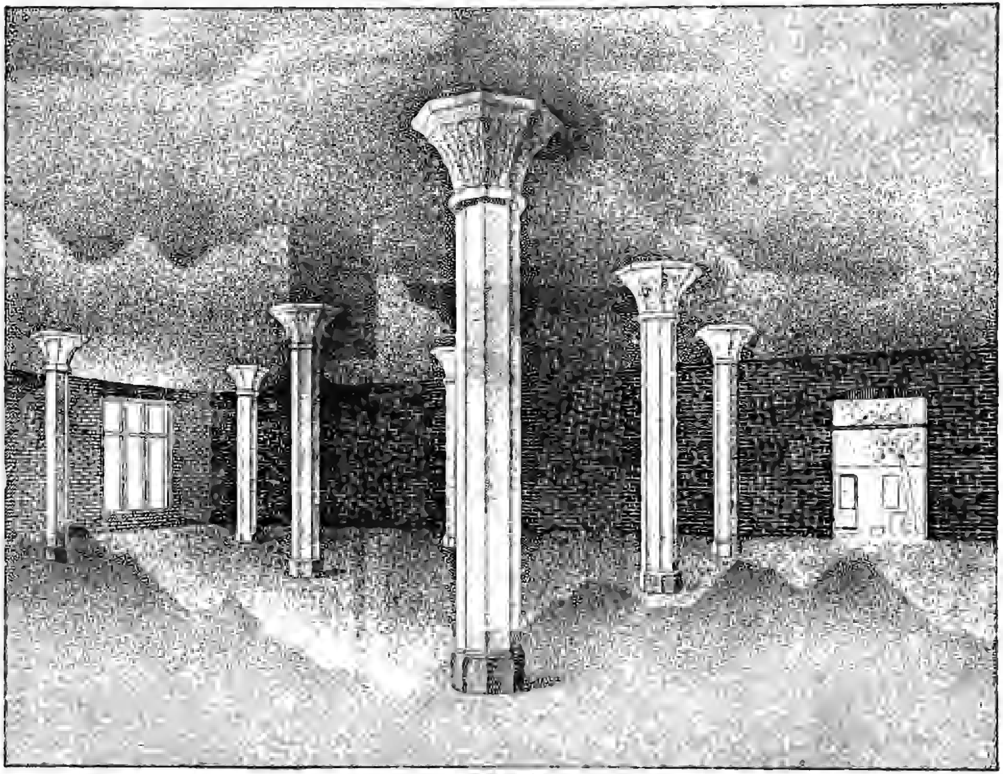


FIG 18

that have a spread cap of heavy reinforcing bars radiating from the column, as illustrated at *a*, Fig. 17. These rods are held radially by circular reinforcing bars *b*, to which they are securely wired, and it is from this cap effect that this system takes its name. By means of this heavy reinforcement, the column is greatly strengthened at the top, and by spreading in this way, it supports a large portion of the floor slab, after the manner of radiating cantilevers. As shown in Fig. 17, the flat slab that covers the column supports is reinforced in several directions with light reinforcing rods.

An interior view of a completed construction with the enlarged column caps is shown in Fig. 18. In this system of construction, panels from 16 feet to 18 feet square and designed to support a live load of as high as 500 pounds to the square foot are possible, while panels as large as 22 feet square may be used for loads of 150 pounds or lighter.

One great advantage of the mushroom system is that it does not require such an elaborate centering for the floor construction as do the other systems. In fact, only a flat centering is necessary between the columns. The columns may be made of the same size throughout the building so that their centering may be used several times, and the centering for the spread cap, if constructed of heavy sheet or cast iron that is properly braced, can be made interchangeable for all floors. It is, however, usually more economical to use columns of different sizes in the several tiers.

The mushroom system of construction is particularly suitable for warehouses, as it permits the storing of material to the ceiling; whereas, ordinarily, the beams and girders take up part of the space.

GIRDER-FRAME, OR FABRICATED, SYSTEMS

25. Unit System.—The system of reinforced-concrete construction known as the **unit system** was invented by E. Perrot, who conceived the idea of tying together the reinforcing bars of one structural element, such as for a beam or a girder, in such a manner as to make it possible to place them in the forms in one complete fabrication, or as a unit.

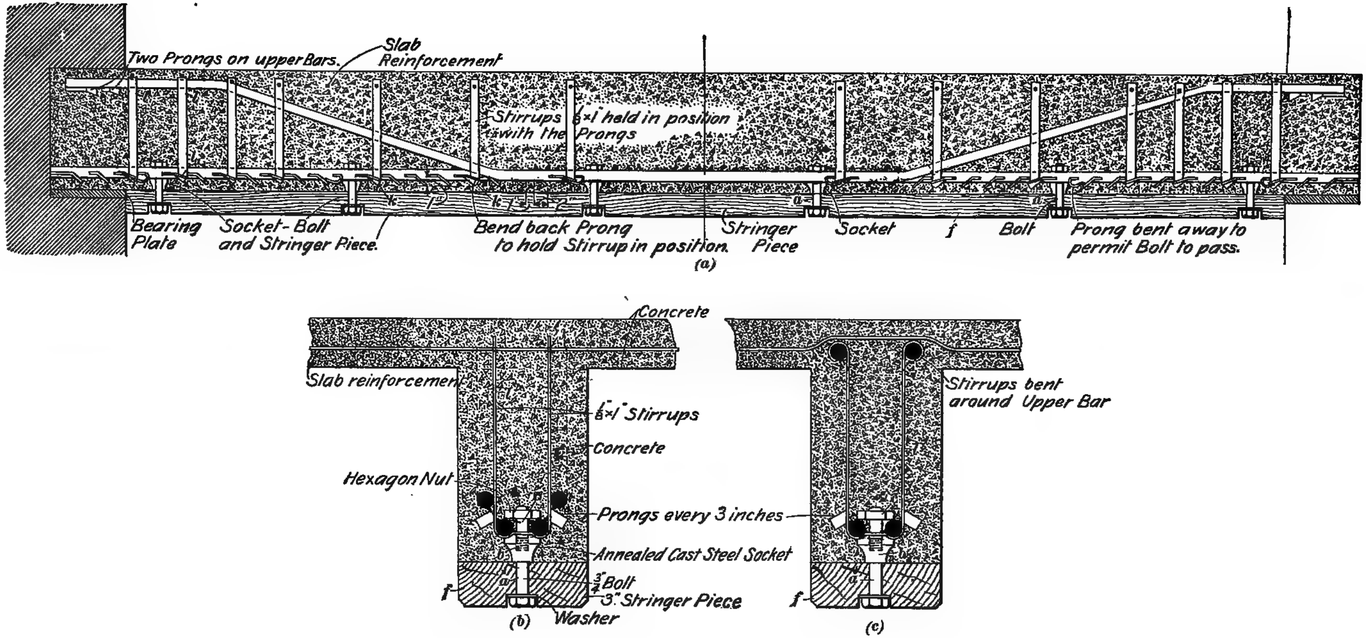

Two Prongs on upper Bars.
Slab Reinforcement
Stirrups $\frac{1}{8}$×1" held in position with the Prongs
Bearing Plate
Socket-Bolt and Stringer Piece.
Bend back Prong to hold Stirrup in position.
Stringer Piece
Socket
f
Bolt
Prong bent away to permit Bolt to pass.
(a)
Concrete
Slab reinforcement
$\frac{1}{8}$×1" Stirrups
Concrete
Hexagon Nut
Prongs every 3 inches
Annealed Cast Steel Socket
$\frac{3}{4}$" Bolt
3" Stringer Piece
Washer
f
(b)
Stirrups bent around Upper Bar
f
(c)

FIG. 19

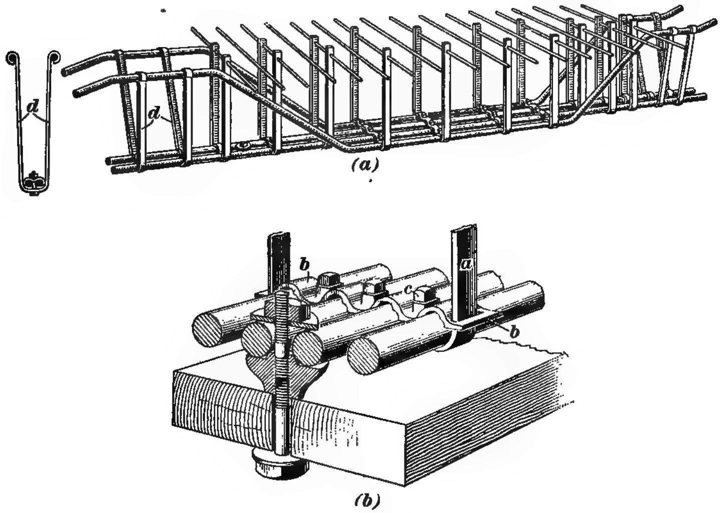
The first type of this reinforcement consisted of angle irons having the necessary net sectional area. These irons were secured by means of clips and bolts, so that all the reinforcement for the beams and girders could be placed at one time. This system was patented and became known as the *unit system of reinforced concrete.*

With the development of the system, the sections known as the *Siamese* and the *Quadbar*, as explained in *Elements of Steel Reinforcement*, were used in the construction of the girder frames. In assembling these rolled sections to make up the necessary reinforcement for particular beams and girders, they were combined in several ways. When the Quadbar section, or the one formed of four round sections connected with webs, was used, it was fabricated into a girder frame, as shown in Fig. 19 (*a*). In using the bar, the outside bulbs of the section were detached for a portion of the length of the bar by shearing close to the section, and the bars were bent upwards, similar to a truss bar. Prongs *k* were formed by shearing the web across and along the axis of the bar and by bending them downwards so as to key with the concrete. Where two Siamese sections were used, they were secured together as shown in the sectional views (*b*) and (*c*); that is, the webs between the bulbs were punched and the two sections held rigidly together by means of a steel socket *b* and a bolt *c*.

The girder frames were delivered formed and ready to set, and when it was required to secure them in the forms, they were fastened by means of the tap bolt *a*, which passed through the bottom of the wooden form board. In this way, the unit girder frame was secured rigidly in the wooden form and at the required distance from the soffit of the concrete beam or girder. After the concrete was in place, the bolt *a* provided a means of securely fastening stringer pieces *f* for the support of the shafting or other installation.

26. In the construction of these girder frames, the ends of the frames were well tied together by means of flat bar-iron stirrups *l* made of 1″ × $\frac{1}{8}$″ material. These stirrups were secured by bending the ends around the truss reinforcing bars and by clamping with the prongs k, as shown at l_1, in view (*a*). The other stirrups throughout the girder were secured by hammering the prongs tight against the bars. All these stirrups projected nearly to the top of the slab, forming the **T** section of the beam, and they were punched at their upper ends, as at j, so as to allow the slab rods to pass through them.

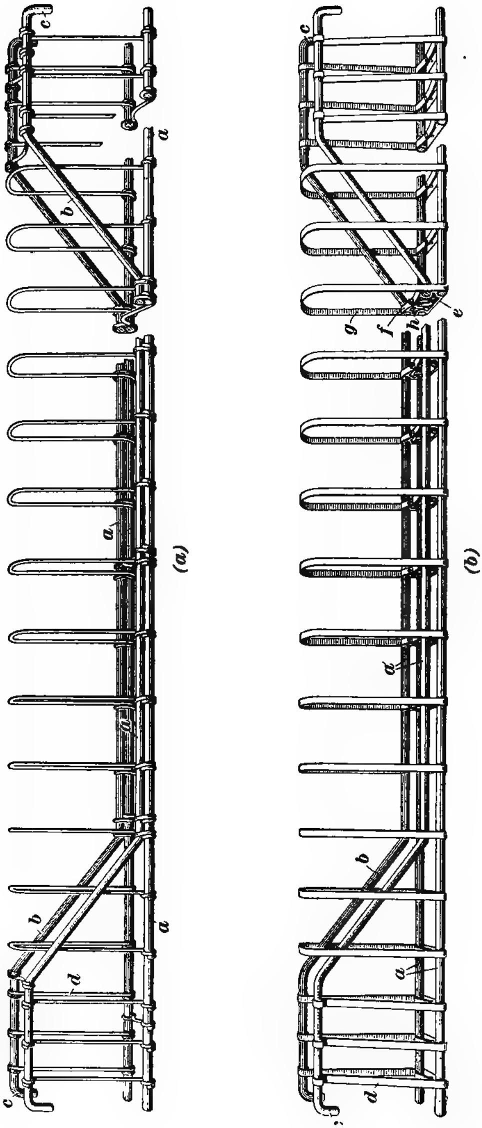
In the construction of the girder frame, the outside rods were bent upwards toward the ends at an angle of about 30° with the horizontal, and at a point far enough from the center not to weaken the beam or girder. By bending the rods up in this manner, additional resistance to shear was introduced throughout the beam or girder; besides, provision was made to take care of the negative bending moments near the bearings. By referring to Fig. 19 (*a*) it will be noted that the web and fin portions of the sections are not sheared through and are not formed into prongs near the center of the beam. This method has one advantage and one disadvantage. First, it will be observed that the entire section of the reinforcement was available at a point where the greatest tensile



(*a*)

(*b*)

FIG. 20



(a) (b)

FIG. 21

stresses occurred. However, it will also be seen that the concrete underneath the section had little key, or bond, with the metal, and a weak point in the construction existed here; that is, since the key, or bond was poor, the fireproofing was insufficient because the concrete was liable to drop off under the severe conditions of a fire.

27. The special rolled bars of the unit system were afterwards abandoned, and the girder frames were made up from any kind of straight or bent bars, as illustrated in Fig. 20. The several bars forming the reinforcement were secured together either by means of flat bar-iron clips or by the means indicated in Fig. 20 (*b*). As shown, the stirrup *a* forms a portion of the clamp for the several bars, and the malleable-iron casting *b* is slotted at the ends for the stirrup to pass through. The stirrup and the casting thus form a clamp for the reinforcing bar, which is secured by means of the bolts *c*. In these girders, the truss bars were held together at the ends by bending the $\frac{1}{8}'' \times 1''$ bar-iron straps *d*, Fig. 20 (*a*), around at the ends, as indicated.

28. The unit girder frame is now made of either plain or bent bars, and the stirrups in the latest development are in the form of an inverted **U**, the construction of the frame being as shown in Fig. 21.

The frame shown in view (*a*) is fabricated, or fastened together, by a shrinking process; that is, each stirrup and tie-rod is hot-shrunk to the main members. This girder frame is designed to be constructed in the shop of the manufacturer, and, consequently, is built to withstand the hard usage of shipment and unloading. The type of frame shown in view (*b*) is designed for use in districts located so far away from the shops of the manufacturer as to make the freight rates for shipping complete frames prohibitory. The frames are so arranged that they may be fastened together with clips, cast separators, and bolts, and any local iron worker should be able to fabricate them from shop drawings supplied by the firm that controls the patents.

In the frames shown in Fig. 21 there are four main reinforcing rods *a*, although the frame may be made up with any number of rods necessary to secure the desired sectional area. The two rods at the top are bent upwards at an angle of about 30° with the horizontal, as shown at *b*, and the ends *c* are turned over so as to add to their grip in the concrete. The stirrups in both frames are shown at *d*, $\frac{3}{8}$-inch round bar iron being used for the stirrups of the frame shown in view (*a*), and light strap, about $\frac{1}{8}$ in. × 1 in., for the stirrups of the construction shown in view (*b*). In the latter frame, the section at the right of the figure shows the reinforcing bars held in the clamping device, which consists of the casting *e*, the clip *f*, and the stirrup *g*. These parts are secured by means of a bolt *h*.

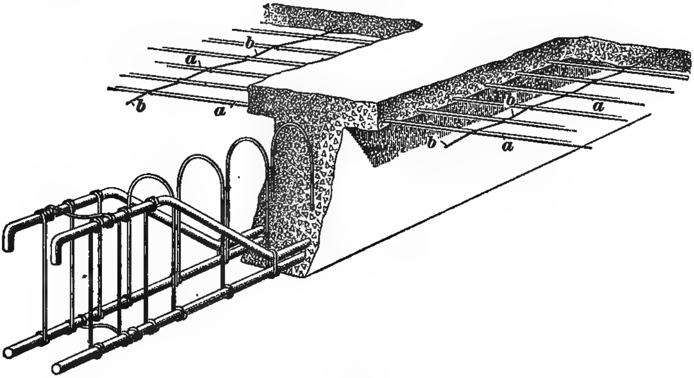


Fig. 22

The position of the girder frame when embedded in the concrete of a floor system is shown in Fig. 22, which also shows the slab rods *a* and the shrinkage rods *b*. The slab rods may either be passed through the loop of the stirrup or be wired to it.

29. An advantage claimed for the unit system is that the reinforcement for each beam or girder can be put together in the shop and be shipped in one piece to the job. By the method of securing the girder frames in the forms, the metal cannot be disturbed in tamping the concrete, and it occupies

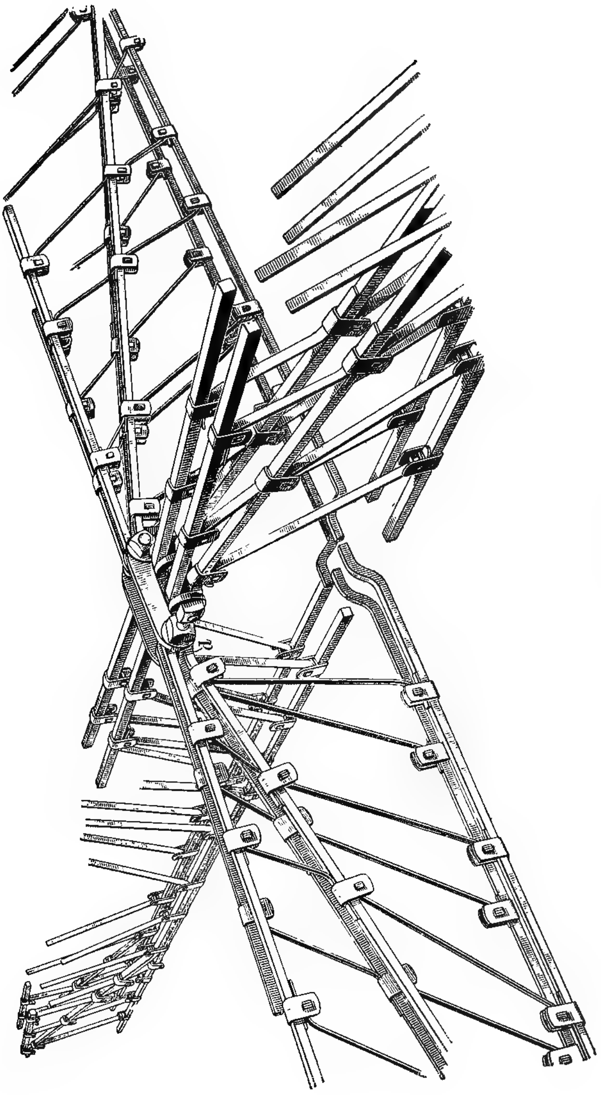


FIG. 23

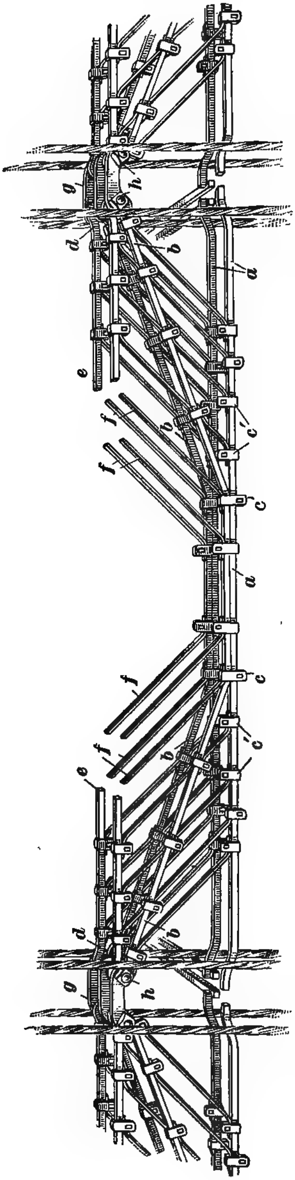


Fig. 24

the exact position intended by the designer. It is practically impossible, where the unit girder frame is employed, to omit any of the principal reinforcing bars in a structural member through carelessness. Some contractors claim that the labor saved in placing the frames in the forms more than offsets the added cost of manufacture, and that the use of these frames insures an actual saving and avoids considerable annoyance and inconvenience on the work.

30. Pin-Connected Girder Frame.—Another type of rod reinforcement built up in the form of a girder frame is illustrated in Fig. 23. This reinforcement is known as the **pin-connected girder frame,** and it is arranged so that the frames forming the reinforcement of the beams and girders may be connected by pins and links at intersections and over column supports. This construction is employed to tie the structure together and to obtain the advantage of a continuous beam, or of one fixed at the ends.

While the general features of the pin-connected girder frame are shown in Fig. 23, its detail construction can best be understood by referring to Fig. 24. In this figure, the main reinforcing bars are illustrated at *a*. The bottom bar runs straight throughout the length of the frame, but it is bent up at the ends over the column supports so that the lower bars of the intersecting girder frame will enter the column level with the bottom rods of the beam reinforcement. The uppermost rod *b* is clamped to the bottom rod by means of clips *c*. This rod is bent in the form of a loop, or eye, for the pin connection at the end, as shown at *d*. This loop is shown more fully in Figs. 23 and 26. The end of the rod at this point is forged by special machinery. By bending the upper rod back in a horizontal direction, as at *e*, Fig. 24, the necessary metal to resist negative bending moment is provided at the points of support. Throughout the length of the girder frame the stirrups *f* are secured and clamped to the main rod reinforcement by the clips *c*′, some of which project downwards sufficiently to act as a support for the rod reinforcement on the bottom board of the form. In this way, the metal is held at the proper distance from the centering.

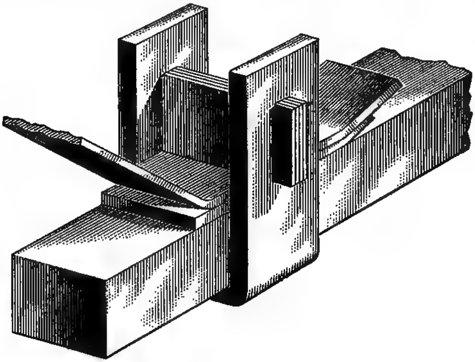


Fig. 25

Links of flat bar iron or steel having holes at the ends, as at *g*, are used as pin connections. These links have a net area in excess of the area of the reinforcing bars, and the pins, or bolts, *h*, which form the pin connection, pass through the eye formed at the end of the reinforcing rod and through the links, the several girder frames being spaced by means of separators.

The clips used to fasten the several bars together and to hold the stirrups to the reinforcing bars are formed of light bar iron, as shown in Fig. 25. They are securely clamped to the main reinforcing bars by means of a key made up of several thin pieces of metal slightly notched at the ends, this key being secured by a thin metal wedge driven and calked tight by a pneumatic hammer. These clips are light in weight, but they hold the stirrups and the several bars together securely.

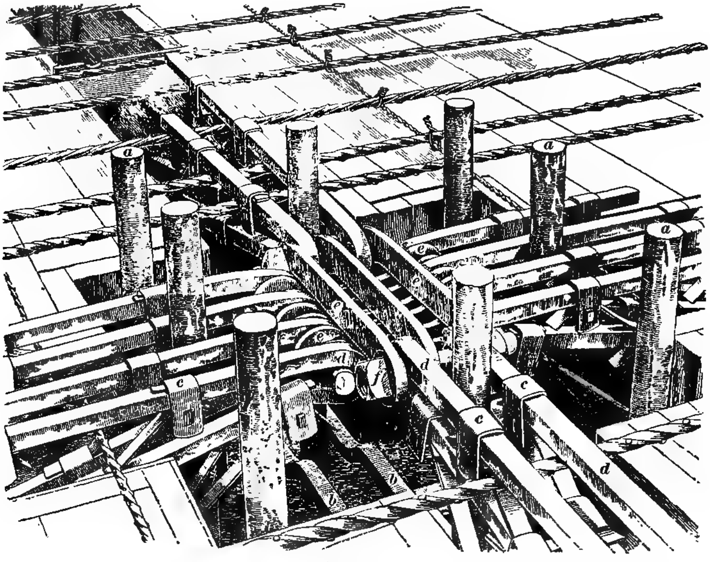


Fig. 26

31. In the construction of this girder frame, mild steel having a tensile strength of from 55,000 to 60,000 pounds per square inch is used. The advantages claimed for this girder frame are similar to those of the unit girder frame, but in addition it is claimed that the pin connection insures absolute continuity of the steel reinforcement over the bearings and thus directly resists negative bending moment. It is also claimed that since the girder frames are built accurately to size, from center to center of pin holes, no girder frame can be misplaced. The pin connections cannot be made unless the proper girder frame for the particular span is used.

To indicate more clearly the placing of the pin-connected girder frames in the forms and their arrangement at the column junction, reference is made to Fig. 26. In this figure, the round bars used for the column reinforcement are shown at *a*; the intersecting beam reinforcements, at *b*; and the camber rods, with their loop for pin connection, and the upper horizontal bars, at *d*. Stirrups projecting downwards from the reinforcement for the resistance of the shear are provided with clips, as shown at *c*. The several connecting links are shown at *e*, and, as will be seen from the figure, one set of links is formed so as to pass under the intersecting links and thus avoid interference at this point. The several bolts forming the pin connection are shown at *f*.

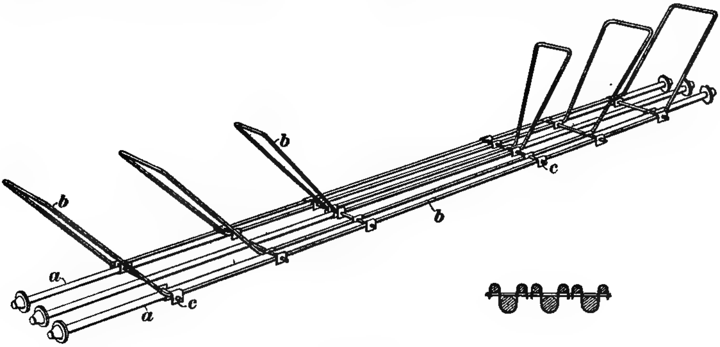


FIG. 27

32. Cummings System of Reinforced Concrete. The system of steel reinforcement shown in Fig. 27 is known as the **Cummings loop-truss girder.** This type of beam-and-girder reinforcement consists of a series of main reinforcing bars *a*, to which are attached a set of smaller bars *b* that turn up at the ends and form a loop. These secondary rods are welded together at the loop and form the stirrups of the frame; besides, they furnish additional reinforcement at the center of the span where the greatest bending moment exists. These stirrups are secured to the main reinforcing rods by means of metal clips *c*, shown separately in Fig. 28. In this figure, the main reinforcing bar is shown at *a*, while the secondary rods, which form the looped stirrups, are shown at *b*. Double **U**-shaped metal clips *c* are bent so as to fit both the secondary rods and the main reinforcing rods. These clips are punched, and by passing a short rod *d* through them the secondary rods are securely held in place to the main rods, or bars. The clips embracing the main rods are sheared and bent down to form a tongue *e* that acts as a support for the girder frame in the forms. These clips also serve to support the main reinforcing bars at the proper distance from the bottom of the form.

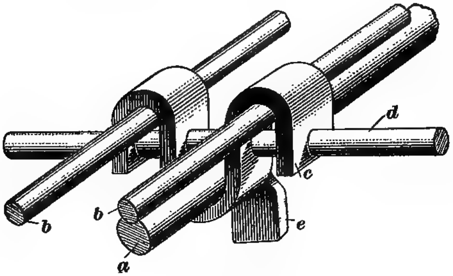


Fig. 28

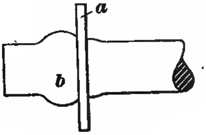


Fig. 29

33. In the Cummings system of construction, it is customary to use round rods, and in order to increase the bond of the rods, the device shown in Fig. 29 is employed. A washer *a* is placed on the ends of the main reinforcing rods, which are upset so as to form a shoulder *b*. This shoulder securely holds the washer in place, and the bond, or anchorage, of the rods in the concrete is thus greatly increased.

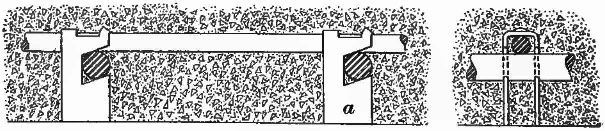


Fig. 30

Another feature of this system of reinforced-concrete construction is the use of a metallic chair lock, to support the

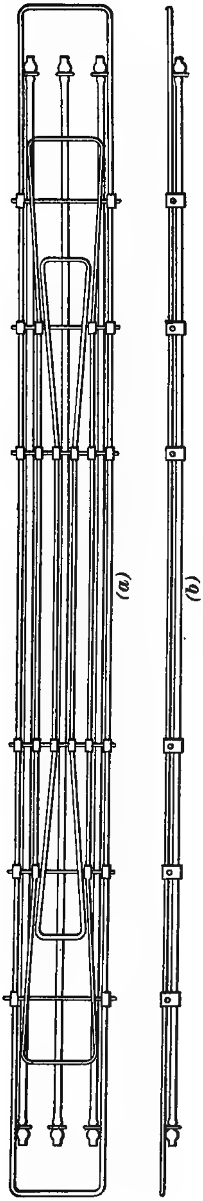


FIG. 31

slab reinforcement. This chair lock is shown in Fig. 30, and consists of a **U**-shaped metallic clip *a* that is cut and bent to the form indicated. This metal clip fastens the main reinforcing rods securely to a transverse, or shrinkage rod, and thus accurately spaces the reinforcing rods. The clip also acts as a support for the reinforcement, holding it at the proper distance from the centering when the concrete is placed.

The Cummings loop-truss girder can be shipped flat by leaving the loops unbent, in which case the bending must be done in the field. The arrangement of the truss girder for shipment is shown in Fig. 31. The complete metallic reinforcement for one beam or girder is shown in view (*a*), while a side view of the same frame is shown in view (*b*).

34. The application of the Cummings loop-truss girder and auxiliary system of reinforcement is shown in Fig. 32 (*a*), which illustrates a complete floor system of this type of construction. The girder reinforcement is shown at *a*, and the beam reinforcement at *c*. The column *b* is termed a *hooped column*, because it is reinforced against bursting by a number of hoops *e*. These hoops are shown more clearly in the enlarged cross-section of the column (*b*). They are properly spaced by means of bars *g*, which are provided with barbs that support the hoops. The hoops, however, may be replaced by winding a metal rod spirally around the reinforcing rods *f*.

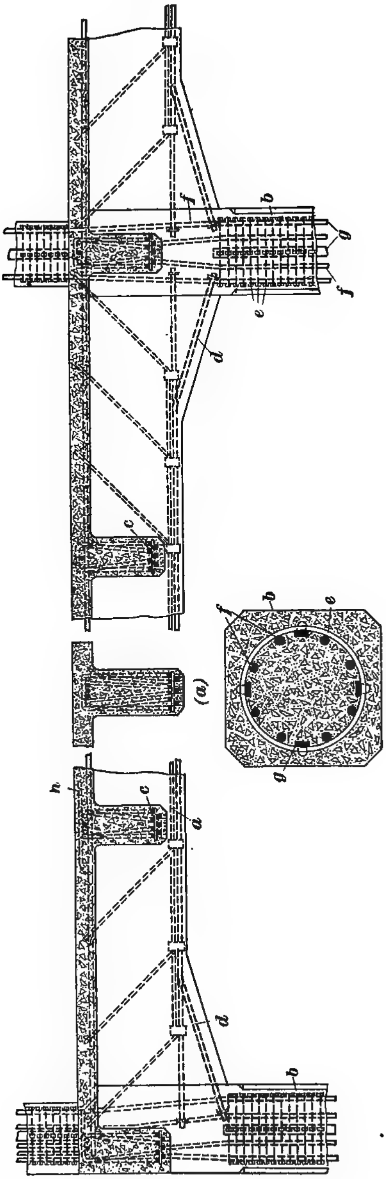


Fig. 32

The interesting features of the illustration are the manner in which the longitudinal column rods are made to converge when passing into the more narrow column above, and the way in which several of the main reinforcing bars are bent downwards to form the reinforcement of the knee brace *d*. Under some conditions, alternate rods of the slab reinforcement are bent upwards, as shown at *h*, in order to provide for negative bending moment in the slab.

35. Universal Reinforcement System.—The type of steel reinforcement known as the **universal reinforcement system** is illustrated in Fig. 33. The longitudinal reinforcement of the column is made of flat bar sections that are slotted on the edge at intervals, as shown

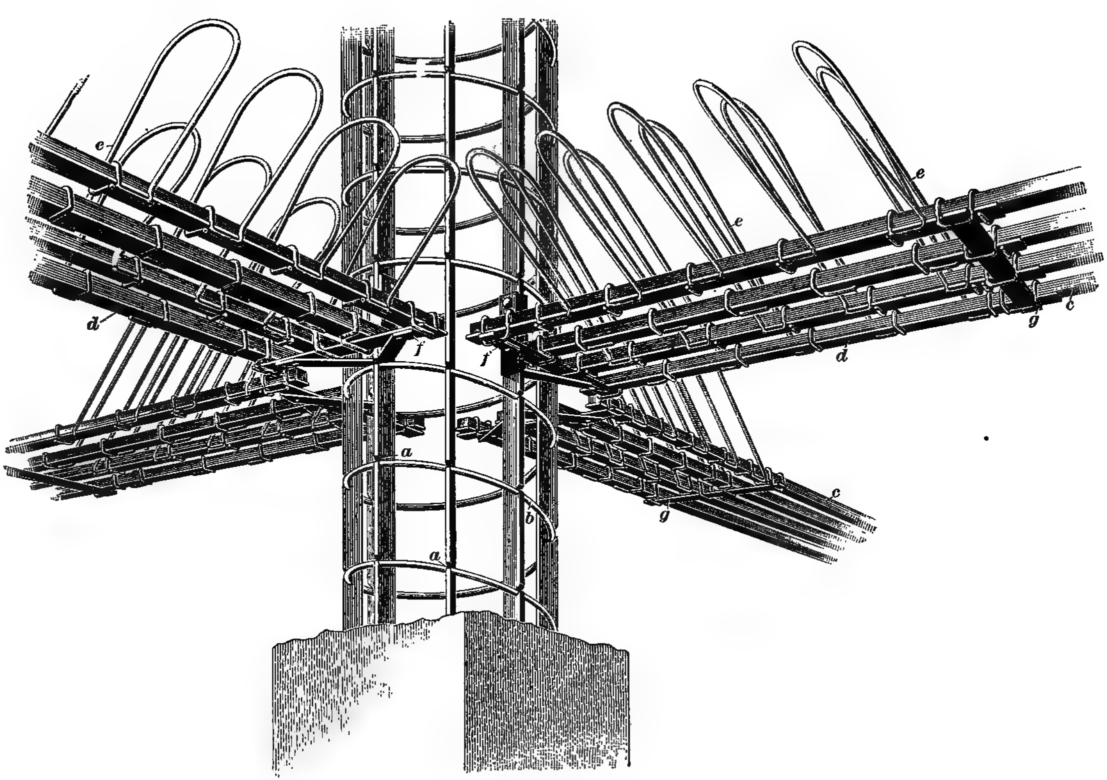


FIG. 33

at *a*. These slots are made at an angle so that they will receive the spiral-wire reinforcement *b*, and also so that the acute-angle edge of the slot can be driven or calked in such a way as to hold the reinforcement in place. The longitudinal bars of the columns are also slotted at the top and the bottom so as to admit rings of metal as spacers. These rings hold the longitudinal reinforcing bars in their proper relative positions.

As shown in the illustration, the main reinforcing bars *c* of the beams or girders are plain square bars, flat bars *d* being used in conjunction with them near the bearings. These flat bars are slotted and are clamped to the main reinforcing bars by the bent iron stirrups *e*, which engage with the slots. The purpose of placing a flat bar near the bearing is to provide a means for fastening the stirrups to the main reinforcing bars. When the stirrups are in place, the edge at one side of each slot is driven, or swaged, down tight, thus clamping the stirrups in place and securing them, together with the flat bar, to the main reinforcing rods. In order that the several main reinforcing bars may be spaced properly and also be supported at their ends by the longitudinal reinforcement of the column, the braced bracket bar *f* is used. Spacing bars *g* are used for the purpose of maintaining the main reinforcing bars parallel throughout their length. The spacing bars are held by means of round bar-iron clamps that engage with the slots in the flat bar-iron cross-piece, or separator. The longitudinal reinforcement of the columns may be spliced by means of ordinary splice plates secured with bolts or rivets.

It is claimed for the universal system of reinforcement that all parts of the construction are securely attached and united, that the main reinforcing bars are held in alinement, and that the stirrups form a mechanical bond for the main reinforcing rods. If desired, the main reinforcement members may be made up of one piece by using a rolled structural **T** section. In such a case, the horizontal portion of the **T** is slotted and the flat bar near the abutments is eliminated. It is claimed that this type of steel reinforcement may be used for the construction of sewers, conduits, subways, and tunnels. It is especially suitable for chimneys, because the vertical main members are slotted and can be extended by

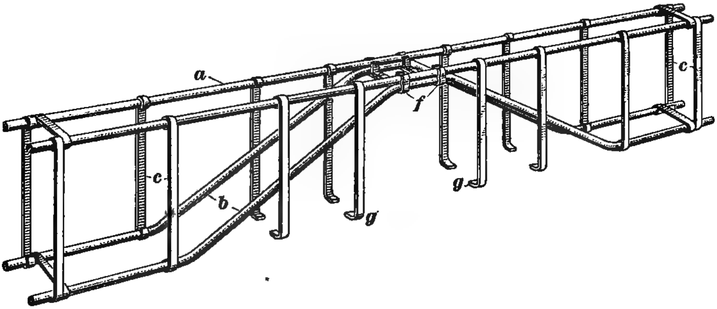


FIG. 34

the splicing system as the work progresses. There is little shop work required in the construction of this type of reinforcement, and the field work is very simple, consisting only of the clinches made in the slots. For this purpose only a hammer and a setting tool are required.

36. Shear-Frame System of Steel Reinforcement. One of the later types of girder-frame reinforcement,

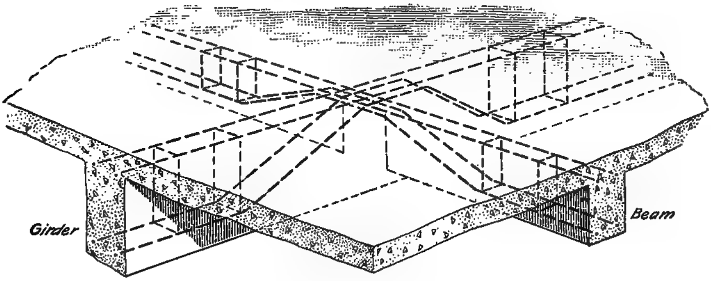


FIG. 35

known as the **shear-frame system,** is employed to resist the negative, or reversed, bending moments that occur at the points of support of monolithic beams and girders.

This type of reinforcement consists of a built-up frame constructed as shown in Fig. 34.

As it will be observed, the shear frame is a complete unit, consisting of straight top rods *a* and bent bottom rods *b*. These two sets of rods are securely clamped together by means of flat bar-iron plates, or clips, *c*, which also act as stirrups. Additional stirrups are provided at *g*, and where the top and bottom rods join, they are firmly fastened together by means of special clamps *f*.

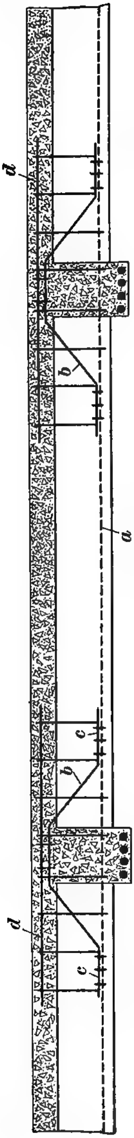


FIG. 36

This type of reinforcement provides a positive tie, or junction, between beams and at the intersection of beams, girders, and columns, insuring against failure at these points by providing resistance to shear and to failure by horizontal or oblique tension cracks. The frame may be used with any type of reinforcement, and is placed at the junction of beams with girders, as shown in Fig. 35, and at the intersection of the girder and column supports. In Fig. 36, the main reinforcing rods of the beams are shown at *a*, while at *b* is shown the shear frame placed in the forms over the girder and beam bearings. Each shear frame engages with, and may be wired to, the main reinforcing rod with the cambered or trussed rods *c*. The straight top rods *d* furnish the necessary resistance to negative bending moments, and at the same time allow the beams to be proportioned by the formula for the bending moment in beams fixed at both ends.

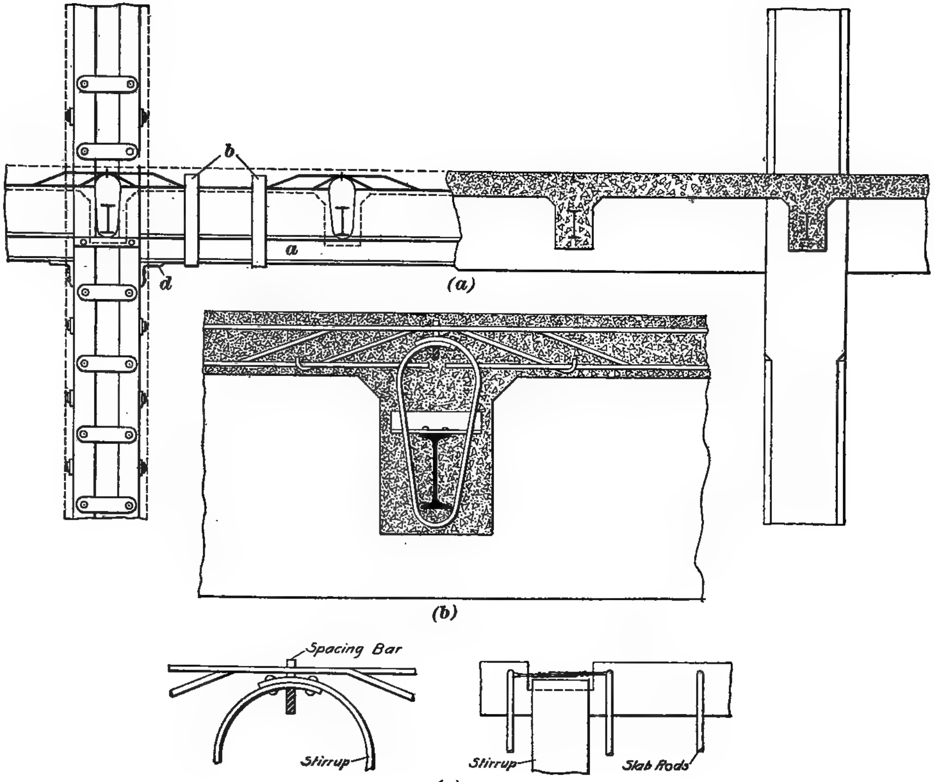
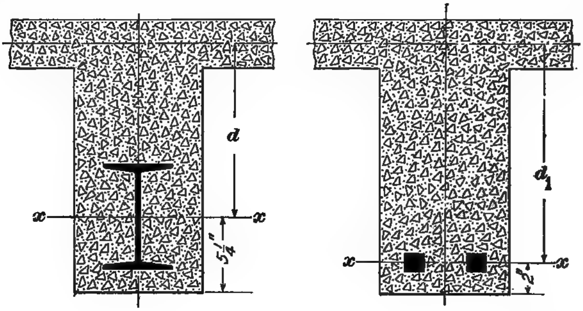


FIG. 37

MISCELLANEOUS SYSTEMS

STEEL-FRAME SYSTEMS

37. Brayton System.—The system of reinforced concrete known as the **Brayton** is illustrated in Fig. 37. In this system, the main reinforcing member of the concrete beam or girder is a standard, rolled-steel, **I**-beam section, and the entire reinforcement, including the steelwork, is put together in a manner similar to the usual steel-frame construction. The structural-steel beams that form the reinforcement of the concrete are designed of sufficiently heavy sections to carry the dead load of the centering and the weight of the concrete. Thus, shores and false work other than the centering need not be used in the erection of the building, and the work can therefore proceed with considerable rapidity and with a minimum amount of carpenter work.



(a) (b)

FIG. 38

The **I** beam forming the main reinforcing member of the concrete beam is shown in view (*a*). As shown at *b*, the stirrups extend beyond the top flange of the **I** beam in the form of loops, and are either riveted through the web or fastened by clipping them to the lower flange, as indicated in the beam cross-section (*b*). The steel **I** beam is supported on a column that is built up of structural-steel angles and is provided with angle brackets *d*. In this way, adequate resistance to direct vertical shear is secured at the points of support. The slab rods are fastened to the stirrups by means of clips and wiring, as shown in detail in view (*c*).

38. The principal deficiency in the Brayton system can be readily seen by referring to Fig. 38. In this figure, which is drawn to scale, are indicated a section of a beam reinforced with rods and one reinforced with an **I** beam, after the manner of the Brayton construction. From view (*a*) it will be noticed that, since the **I** beam has considerable depth, the center of the steel section on the line $x\ x$ is some distance from the bottom, or soffit, of the beam; whereas, in view (*b*), the reinforcing rods are placed within a distance of 2 inches from the bottom of the beam. As these rods occupy little space in height, the center of effort in the steel-bar reinforcement is at a much greater distance from the upper part, or compressive section, of the beam. This difference is shown by the two distances d and d_1. From these it is evident that the leverage of the steel reinforcement in the Brayton system is much less than in any of the rod systems. Therefore, more metal must be used in order to provide the required reinforcement.

39. While the Brayton and similar systems provide a thoroughly rigid structure, they cannot be erected so cheaply as can the loose-rod systems, and for this reason they are not being used to the same extent. It has also been found that owing to the usual delays in getting structural rolled shapes, reinforced-concrete buildings can be built quicker where rods are used for reinforcing, although, usually, time is gained in erection by using structural steel.

Another objection to the Brayton system is that the great depth of the web of the **I** beam and its peculiar shape tend to deteriorate the bond between the concrete and steel. An **I**-beam section cannot possess the same skin friction and bond as a number of rods surrounded by concrete.

40. Standard System of Concrete-Steel Construction.—In Fig. 39 is shown a type of reinforced-concrete construction that resembles the Brayton system.

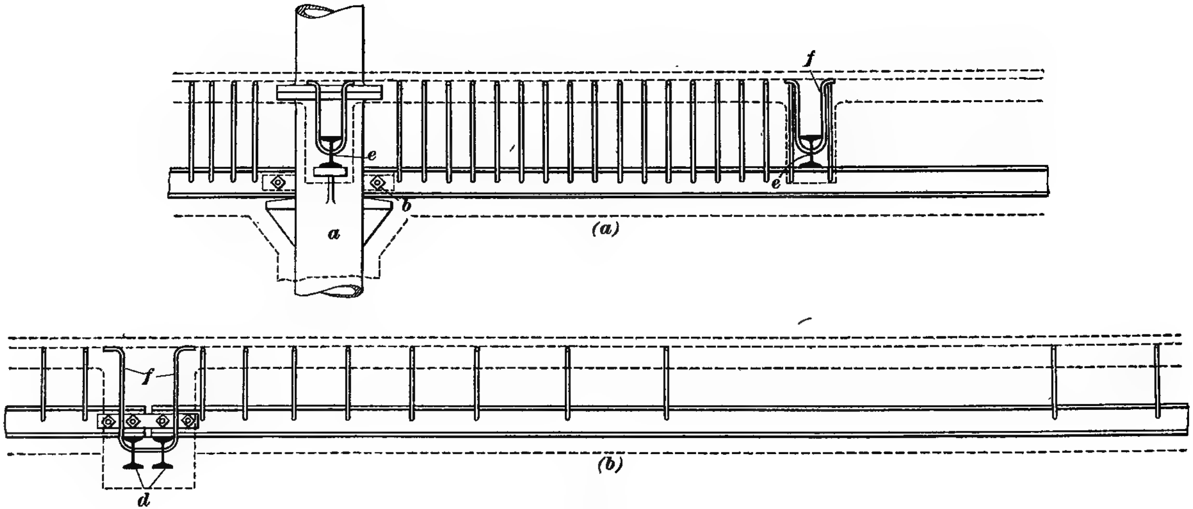


FIG. 39

In this system, which is known as **system M, standard concrete-steel construction,** the columns *a* are made either of structural steel or of cast iron. The main reinforcement of the beams and girders consists of structural rolled shapes, which are usually simple **I** beams, since these are the most available. The columns are erected and plumbed first, and then the **I** beams forming the reinforcement of the girders and those forming the reinforcement of the beams resting upon the columns are set in place upon the lugs and brackets and bolted to the column lugs, as shown at *b*. The reinforcement of the beams that intersect the girder rests upon the **I** beam that acts as the girder reinforcement, as at *e*, and is thus supported.

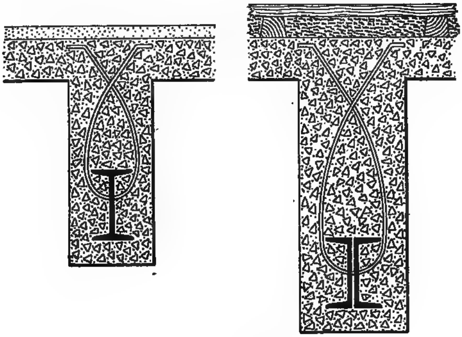
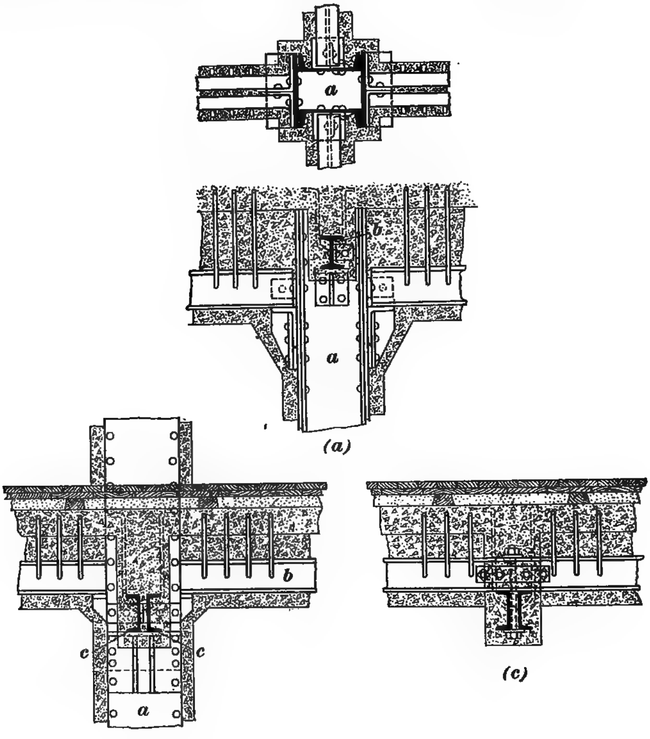


FIG. 40

In the construction shown in the figure, the girder reinforcement consists of two 4-inch **I** beams weighing 10½ pounds per foot. These beams are shown at *d* in the sectional view (*b*). The beam reinforcement *e* consists of a similar **I** beam, as shown in view (*a*). The stirrups in this type of construction are of ⅜-inch round iron or steel, and they pass through holes punched in the center of the web of the **I** beams. These stirrups are either bent up in **U** form, as shown at *f*, or they are bent across and embedded in the concrete in the manner shown in Fig. 40.

41. It is claimed that this type of reinforcement costs less than structural-steel construction and, besides, possesses many of its advantages. The steel beams are of sufficient strength to carry the dead-weight of the centering and the concrete. They likewise act as a positive tie between the columns of the structure, and since the construction is erected in the same manner as steel construction, with every piece arranged to bolt in place, there can be no omission of steel reinforcement through carelessness during the operation of building. It is also evident that the steel will be accurately located in the forms, and that the stirrups will have a positive mechanical bond with the steel reinforcement of the beams and girders. The primary advantage of the system is that the centering can be entirely supported from the steelwork



(a)

(b)

(c)

Fig. 41

forming the reinforcement, without the addition of the many supports, or props, necessary in the usual type of reinforced-concrete construction.

With this method of construction the entire steel fabrication can be erected and the work of concreting can proceed simultaneously on several floors. This method of procedure cannot be employed in the erection of a reinforced-concrete building where the centering for the uppermost floors depends for support on the concrete already set in the floors below.

42. In Fig. 41 is shown details of the standard concrete-steel construction used in conjunction with the structural-steel column. In view (*a*), the column *a* is of box section, being built up of channel irons with cover-plates and provided with the necessary angle brackets and clips for securing the girder and beam reinforcement to it. As will be noted, the beam reinforcement *b* consists of a single **I** beam, while the girder reinforcement, shown in view (*b*), is composed of two channel irons *c* arranged back to back. These channels have sufficient space between them to allow the concrete to flow in and to permit the beam reinforcement to be secured to the girder reinforcement, which is done by passing a bolt between the channel irons of the girder and the abutting ends of the beam reinforcement, as shown in view (*c*). In this system of

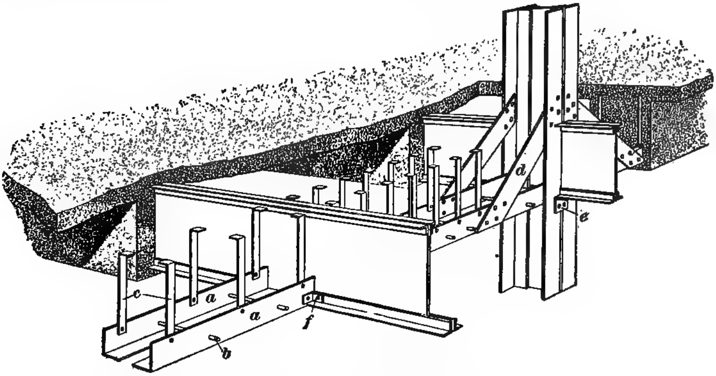


FIG. 42

construction, all beam reinforcement is tied across the girder with bar-iron splice plates, which are bolted with two bolts at the end of each beam. In these views, the concrete is shown removed from the parts lying beyond those in section in order that the method of construction may be shown more clearly.

43. Conard System.—Another system of reinforced-concrete construction in which structural shapes that are fastened together are used for the steel reinforcement, is shown in Fig. 42. This system, which is named after the patentee, is known as the **Conard system.**

Any structural-steel rolled shapes, such as channels, **I** beams, or angles, may be employed as the main reinforcing rods in this system. The beam construction shown consists of two rolled-steel angles *a*, with their vertical legs at the outside of the beam. Through these vertical legs ¾-inch round bar irons *b* are passed, being secured in place by riveting or by calking. These bar irons are known as *transmission bars* and act as a spacing device for the rolled shapes. They also provide additional bond in the concrete. The necessary stirrups *c* consist of flat bar iron, with or without angle clips at the upper end, and are secured at the lower end to the main reinforcing sections by means of rivets.

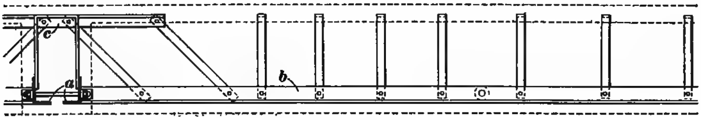


FIG. 43

Another feature of this system is the main stirrup, or shear bar, *d*, which is riveted to the reinforcing bars and to the structural-steel column core. It will be observed that the ends of the reinforcement of the girders are secured to the column core by means of angle clips *e*, and that the reinforcement of the beams is secured to the reinforcement of the girders at the intersection by other angle clips *f*.

The negative bending moment at the point where the beams and girders of this construction meet is provided for by the method shown in Fig. 43. In this figure, the reinforcement of the main girders is shown at *a* and the reinforcement of the beams at *b*. The two end stirrups of the beams are connected at the top by an angle iron *c*, which furnishes the necessary reinforcing at the top to provide for the negative moment.

In Fig. 44 are shown the ways in which several standard sections may be used in conjunction with the flat bar-iron stirrup.

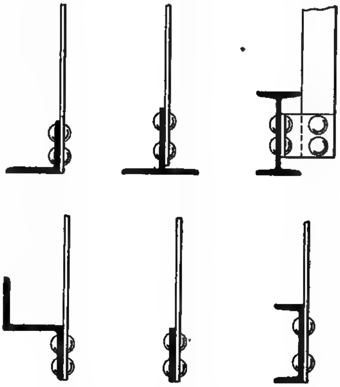


FIG. 44

The disadvantage of using a system of this kind is that the bond between the concrete and the structural-steel section is not so good as that between concrete and round or deformed bars. Also, since there are always broad, flat surfaces of metal reinforcement at or near the bottom of beams or girders, the concrete is likely to lack adhesion, and under conditions of vibration or fire, it will crack and break off, thus exposing the metal. It is doubtful whether systems of reinforcement consisting of rolled-steel shapes can compete with plain round, square, twisted, or deformed bars, because their prices per pound, as well as the cost of the shop work that is usually required, are greater.

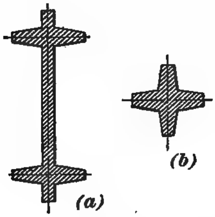


FIG. 45

44. Columbian Reinforced - Concrete System.—The **Columbian system** is used largely for fireproof floors in structural-steel buildings, and when the Columbian bars are used in conjunction with steel I-beam girders, this system of fireproof construction can hardly be included with types of reinforced-concrete construction.

However, the Columbian bar has been adapted to the construction of buildings composed entirely of reinforced concrete. The bar of the section illustrated in Fig. 45 (*a*) can readily be used in the same manner as any rolled-steel section for the reinforcement of concrete, while the smaller sections, as in (*b*), may be employed to reinforce columns, to augment the main reinforcing bars and make up the necessary area, or to serve as truss bars or any other secondary reinforcement.

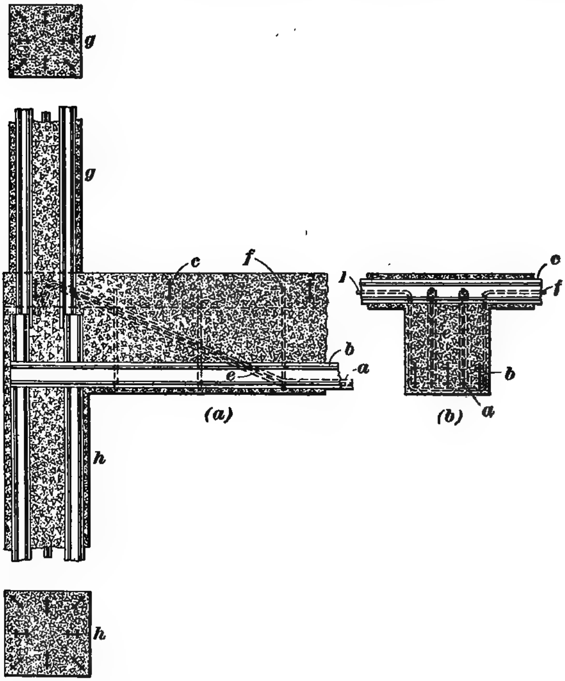


Fig. 46

In Fig. 46 is illustrated the Columbian system of reinforced-concrete construction, (*a*) being an elevation of a column and girder, and (*b*) a cross-section through the girder. In the latter view, the girder is reinforced by the three bars *b* of the usual Columbian section, and between these, as shown in view (*a*), rods or bars *a* of smaller size are bent up, as at *e*, to form truss bars. The slab joining the girder is also reinforced with the Columbian bars *c*, and the several bars reinforcing the girder have a bearing within the column. The truss rods or bars *e*, which are bent upwards at the bearing in order to provide resistance against the negative moment, are hooked at the ends so as to obtain a greater hold, or bond, in the concrete. In order to provide reinforcement to resist the horizontal stresses in the girder, stirrups *f* made of either round or square bars are used. These stirrups are in the form of an inverted **U**, being bent square at the bottom and having the ends turned outwards so as to enter the slab. The bars of this system can be riveted or bolted to the structural-steel column cores by means of angle clips, and thus assist in forming a rigidly connected steel-frame system. The elevation (*a*) shows the junction of two columns *g* and *h* that differ in width.

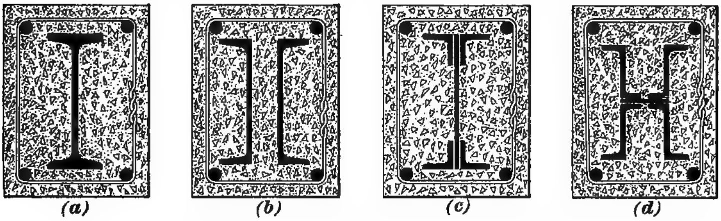


FIG. 47

45. Steel Construction With Reinforced Concrete. Reinforced concrete is sometimes used with steel-frame construction for the erection of sheds and one-story mill buildings. In such constructions, the structural-steel framework, instead of being covered with the usual corrugated galvanized iron, is made more enduring by using concrete roof slabs; also, the building is partly fireproofed by surrounding the steel columns with concrete.

The typical details of this type of construction are illustrated in Figs. 47 and 48. In Fig. 47 are four views showing types of columns embedded in concrete. Columns made in this manner are protected from corrosion and furnish some fire resistance to the main supporting members. Fig. 48

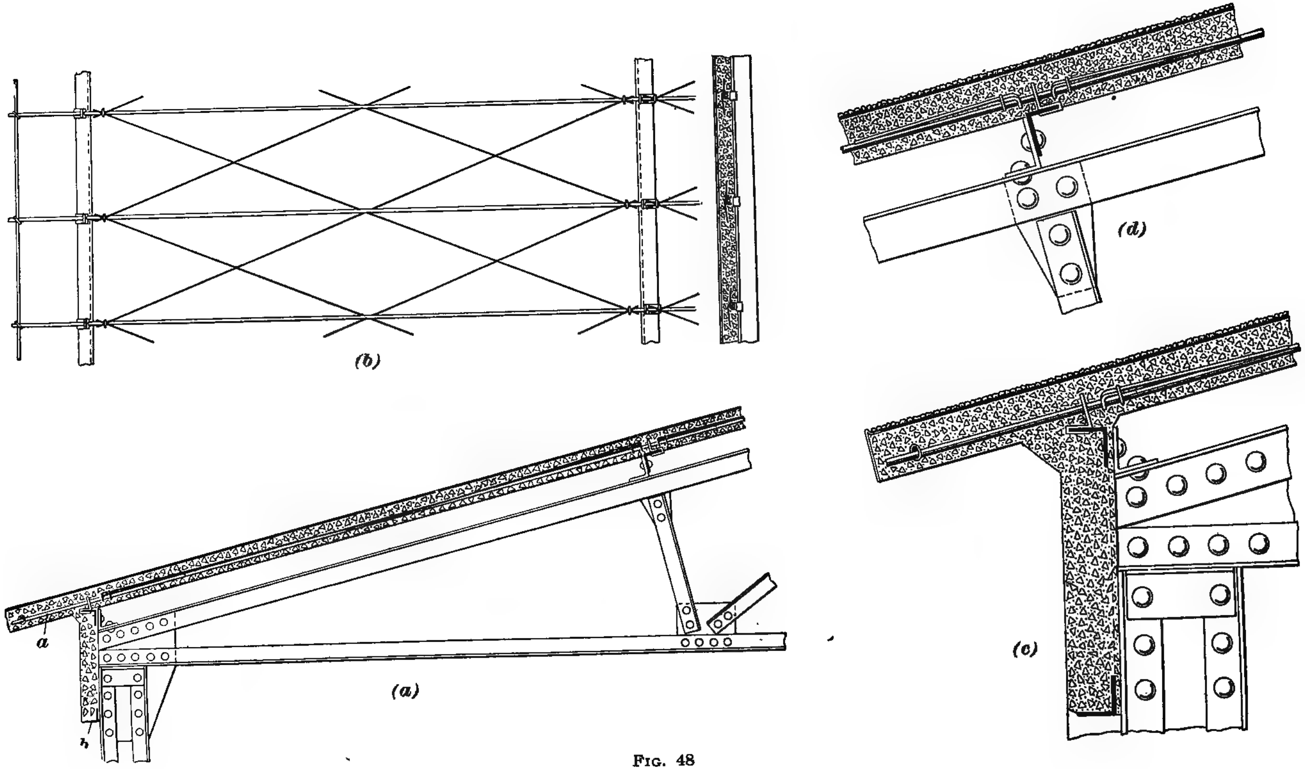


FIG. 48

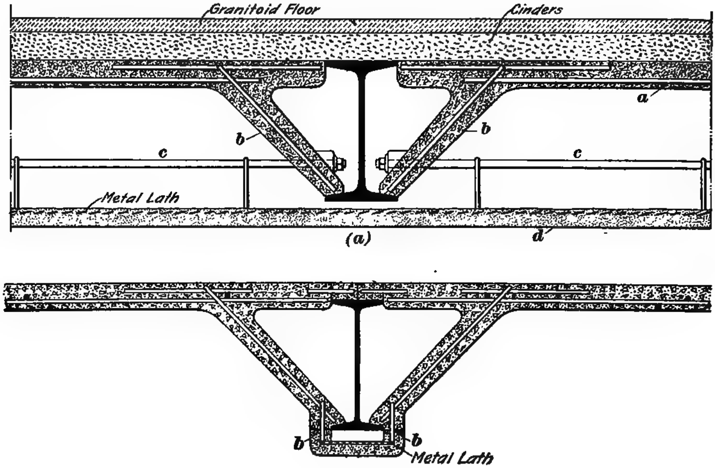
shows the details of the roof construction for this type of building, as proposed by the American Concrete Steel Company. In view (*a*) is shown the steel truss, together with the steel angle purlins that support a reinforced-concrete roof slab. This slab is made impervious by covering it with slag roofing. The main reinforcing bars *a* extend in such a manner as to reinforce the overhang of the roof. The fascia underneath the soffit of the overhang is a 3-inch slab of concrete, and it is supported by the angle purlin at the eave and by the angle string *b*.

In the system of the American Concrete Steel Company, it is the custom to tie together the main reinforcing bars with ¼-inch steel rods. These are hooked over and wired to the main slab reinforcement, as shown in Fig. 48 (*b*). In Fig. 48 (*c*), is shown an enlarged section at the edge of the roof construction, and in (*d*), an enlarged section of the slab supported by the purlin. These details show the position and method of fastening the oblique bonding wires typical of the American Concrete Steel Company's systems.

LINTEL, OR SELF-CENTERING, SYSTEMS

46. Thacher Floor System.—As one of the great items of expense in reinforced-concrete floor construction is the cost of the wooden centering, several systems have been devised to avoid its use. In these systems, reinforced-concrete units are manufactured in suitable molds and in sizes convenient to handle. After the concrete has set for a month or six weeks, it is sufficiently strong to ship and to place in the construction.

One of these reinforced-concrete systems is shown in Fig. 49. View (*a*) represents a floor system in which the principal members consists of 12-inch I beams. Resting on the lower flange of these beams are the oblique legs *b* of the concrete centering *a*. As there is considerable tendency for the leg, or side, *b* to spread under the load, legs of adjoining sections are tied together with ½-inch tie-rods *c*. These tie-rods also act as a support for the suspended ceiling *d*. This ceiling, besides forming a finish for the room beneath, furnishes protection for the steel beam.



(b)

Fig. 49

A similar type of centering, but one that provides for the protection of the steel I beam without the use of a suspended ceiling, is shown in Fig. 49 (*b*). In this case, the soffit of the I beam is protected by plastering between the two projecting ends of the centering, as shown at *b*.

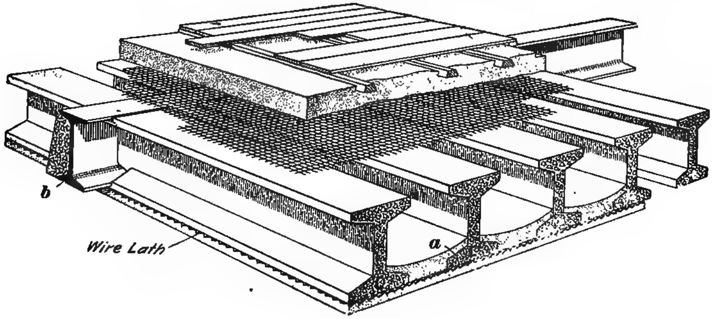


Fig. 50

From both of these views, it will be observed that the reinforcing rods, or bars, are arranged so as to resist the stresses to which the concrete centering is usually subjected. The type of centering shown is designed to sustain a live load of 150 pounds per square foot on 5-foot spans, with a factor of safety of 5.

47. Standard Concrete-Steel Centering, System B. Another type of reinforced-concrete lintel, or centering, is illustrated in Fig. 50. This construction, known as the **Standard concrete-steel centering, system B,** is a product of the Standard Concrete Steel Company. In the construction illustrated, the reinforced-concrete lintels *a* of I section are supported on the steel-beam girders *b*. These lintels are not cast in place, but are made at any convenient locality and then hoisted in place. Over the top of the reinforced lintels is laid a woven-wire mesh, which acts as a centering for the cinder-concrete fill and forms the base for the cement or tile floor construction, or the embedment for the sleepers in case a wood-finished floor is to be used. This type of construction is suitable for very light floor loads and for spans of 7 feet. The ceiling is finished flat with cement plaster, expanded metal or woven wire being placed on the bottom of the concrete lintels so as to form a lath. The cement plaster acts as fireproofing for the bottom flange of the steel beams or girders.

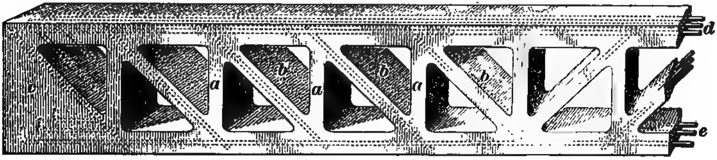


FIG. 51

48. Visintini System.—Another system of lintel construction is the **Visintini system,** controlled in the United States by the Concrete Steel Engineering Company. A general view of a girder constructed according to this system is shown in Fig. 51. The girders can be made of any width and breadth required to withstand the stresses. In

France, the columns of buildings are also built by this system, but in the United States solid columns have been preferred. The beams are made along the lines of latticed steel girders. In light work, the vertical webs *a*, which are in compression, are not always reinforced, but the slanting webs *b*, which are in tension, always have steel in them. What may be termed the upper and lower chords contain each a set of three reinforcing rods, as shown at *d* and *e*, respectively. The beams may be made in any convenient location and put in place on the job afterwards. When solid reinforced-concrete columns are used with this system, these columns are cast in place and are provided with brackets properly designed against shear to hold the principal girders. These girders are then hoisted on to these brackets. Part of the end of the girder is made solid, as shown at *c*, in order to resist the stresses in shear and direct compression due to the reaction of the support.

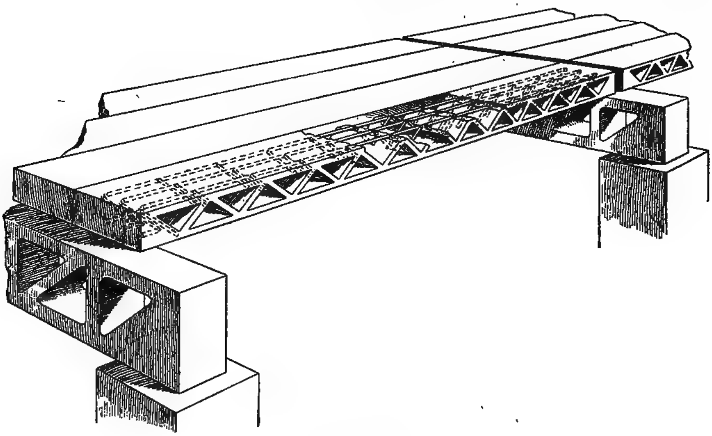


FIG. 52

Entire floors may be made of light beams of this system, as shown in Fig. 52. These beams are also made solid at each end so as to supply sufficient resistance against shear. The space between the abutting ends of beams over each girder is filled with concrete after the beams are placed.

The Visintini system possesses the advantage of being strong and light. It requires no forms or timberwork except for the columns, and the construction requires less concrete than solid beams. However, the beams are not so easy to cast as solid beams and the structure is not monolithic; that is, built in one piece.

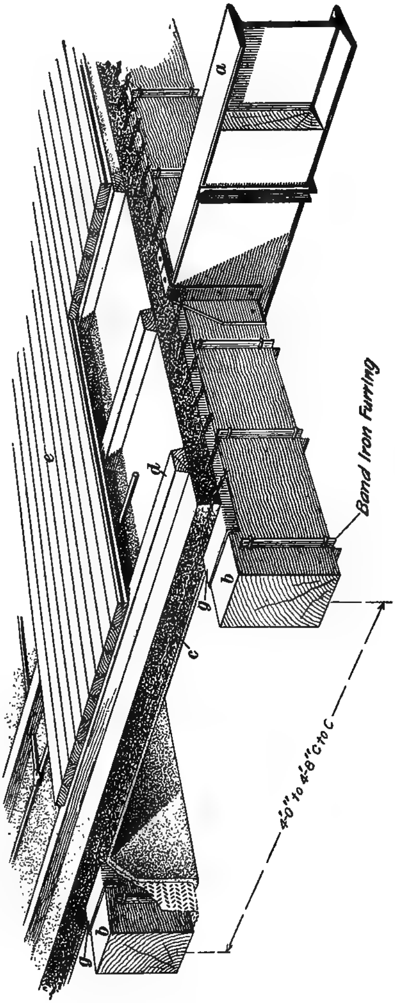


FIG. 53

COMPOSITE SYSTEM

49. There is a type of construction proposed and controlled under the Hinchman-Renton patents that combines reinforced-concrete construction with the usual mill, or slow-burning, construction. This type of construction is illustrated in Fig. 53. The main girders *a* consist of steel **I** beams that support heavy floor timbers *b*, and the slab construction, instead of being made up of the usual 3- or 4-inch tongued-and-grooved planking, is a 3- or 4-inch slab of cinder concrete that is reinforced with fence wire *c*. Resting on top of the concrete slab are sleepers, or nailing pieces, *d*. These are spiked to the cinder concrete, and when filled in between, form a solid nailing for the finished floor *e*. In order that the timbers may be preserved where they come in contact with the cinder concrete, they are protected on the top with a sheet-metal cap *g*. It is also proposed to protect the wooden beams by covering them with metallic furring, lath, and plaster. It is claimed that where ordinary lime mortar is used for the plastering there is little danger of dry rot. The steel beams could likewise be fireproofed with metallic lath and plaster.

The advantages of this system are that the floor slab has greater stiffness and strength and there is greater fire resistance between floors; besides, such floor construction will not transmit sound so readily as a plank floor. The disadvantages exist in the increased dead-weight due to the floor-slab construction, and in the danger of decay likely to follow the enclosing of wooden beams that are not thoroughly seasoned. The increased dead load of the floor slab over planking is serious if the spans of the wooden beams are long, as it is sometimes difficult to get timbers of sufficient size for heavy construction where the spans exceed 20 feet, and a minimum dead load is consequently desirable.

CONCRETE BEAM AND COLUMN DESIGN

PLAIN CONCRETE

CONCRETE BEAMS

INTRODUCTION

1. As the design of stone beams and columns is like that of plain-concrete beams and columns, they will all be considered in the first part of this Section.

Both stone and concrete, particularly the latter, are weak in resisting bending stresses. This weakness is due not to the crushing at the top of a beam but to the lack of tensile qualities, which allows the beam to start its fracture on the under side. Thus, a beam of concrete or stone never breaks as shown in Fig. 1 (*a*), but always as shown in (*b*). Fortunately, stone beams are seldom used, and plain-concrete beams are of still less frequent occurrence. The use of stone or plain-concrete beams is confined almost exclusively to lintels over door and window openings. In such cases, the size of the beam is governed by the architectural effect that it is desired to obtain, and the beam will usually be

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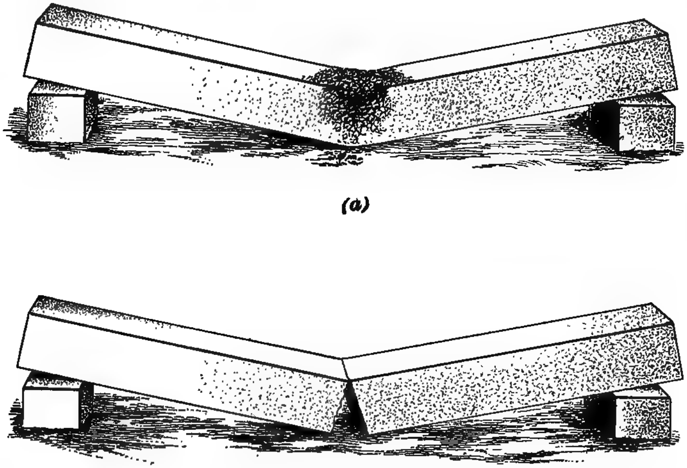
much larger than is necessary for mere strength. Nevertheless, it is sometimes necessary to calculate the strength of such beams, and for this reason the engineer should understand the methods employed in their design.



(a)

(b)

FIG. 1

METHOD OF DESIGN

2. Stone and plain-concrete beams are designed by exactly the same method as any other kind of beam, as one of wood, steel, or the like. The formula employed is as follows:

$$M = \frac{s\,I}{c} = S\,s,$$

in which M = bending moment;
s = unit stress produced;
I = moment of inertia;
c = half the depth of the beam;
S = section modulus.

In a beam of rectangular section, $S = \frac{b\,d^2}{6}$, in which b is the breadth of the beam and d is its depth, both measured in inches.

3. Stone Beams.—As was mentioned, the formula given in the preceding article can be used in the design of stone beams. The only quantity that remains to be known is the modulus of rupture. This value for various kinds of stones and other materials is given in Table I.

TABLE I

MODULI OF RUPTURE OF VARIOUS MINERALS

Material	Modulus of Rupture
Slate	5,000
Glass	3,000
Bluestone flagging	2,250
Marble, white, Italian	2,090
Marble, white, Vermont	1,850
Marble, gray, Vermont	1,260
Granite, Quincy, Massachusetts	1,800
Granite, New York	1,800
Granite, Connecticut	1,500
Sandstone, Massachusetts	1,080
Sandstone, Middletown, Connecticut	1,000
Sandstone, Ohio	500
Freestone, Little Falls, New York	1,730
Freestone, Belleville, New Jersey	1,440
Freestone, Dorchester, Massachusetts	800
Freestone, Hubeginy	650
Freestone, Caen, Normandy	450
Limestone, average value	1,500
Brick, common or Philadelphia pressed	600
Brick, best hard	800
Rubble masonry in cement	200

With the values in this table, a factor of safety of from 10 to 20 is usually employed when problems dealing with the safe load instead of the ultimate breaking load are being solved.

Example 1.—A stone beam is 6 inches broad and 10 inches deep, and it sustains a load of 1,000 pounds in the center of its span. The span is 4 feet. What stress is produced? Neglect the weight of the beam.

Solution.—In this case, $b = 6$ in. and $d = 10$ in. Therefore,

$$S = \frac{b\,d^2}{6} = \frac{6 \times 10^2}{6} = 100$$

Also, $l = 4 \times 12 = 48$ in. and $W = 1{,}000$ lb. Therefore,

$$M = \frac{W\,l}{4} = \frac{1{,}000 \times 48}{4} = 12{,}000 \text{ in.-lb.}$$

Substituting these values in the ordinary formula $M = S\,s$, then $12{,}000 = 100\,s$. Therefore, $s = 12{,}000 \div 100 = 120$ lb. per sq. in. Ans.

Example 2.—Design a Quincy granite lintel 6 inches wide on a 5-foot span to carry a total uniform load of 300 pounds per foot with a factor of safety of 10.

Solution.—In this example, $W = 300 \times 5 = 1{,}500$ lb. and $l = 5 \times 12 = 60$ in. Therefore,

$$M = \frac{W\,l}{8} = \frac{1{,}500 \times 60}{8} = 11{,}250 \text{ in.-lb.}$$

According to Table I, the modulus of rupture for Quincy granite is 1,800, and if a factor of safety of 10 is used, the value of s is $1{,}800 \div 10 = 180$ lb. per sq. in. Therefore,

$$S = \frac{b\,d^2}{6} = \frac{6\,d^2}{6} = d^2$$

Substituting these quantities in the formula $M = S\,s$, then $11{,}250 = d^2 \times 180$. Thus, $d^2 = 62.5$ and $d = 7.906$, say 8, in. Ans.

Example 3.—A bluestone slab must support 200 pounds per square foot, including its own weight, with a factor of safety of 15. The supports are 10 feet apart. How thick will the slab be?

Solution.—Any width of slab may be assumed. It will be convenient to assume one that is to be 12 in. wide. Then, $W = 10 \times 200 = 2{,}000$ lb. and $l = 120$ in. Therefore,

$$M = \frac{W\,l}{8} = \frac{2{,}000 \times 120}{8} = 30{,}000 \text{ in.-lb.}$$

According to Table I, the modulus of rupture of bluestone is 2,250; then, $s = 2{,}250 \div 15 = 150$ lb. per sq. in. Also,

$$S = \frac{b\,d^2}{6} = \frac{12\,d^2}{6} = 2\,d^2$$

Substituting these values in the formula $M = S\,s$, then $30{,}000 = 2d^2 \times 150$. Therefore, $d^2 = 100$ and $d = 10$ in. Ans.

EXAMPLES FOR PRACTICE

1. A limestone beam is on a span of 4 feet 2 inches. If the beam is 6 inches deep and 4 inches wide, and an allowable stress of 100 pounds per square inch is employed, what total load per foot will it carry?
Ans. 92.16 lb.

2. A slate beam is 10 inches wide and 12 inches deep and is on a span of 4 feet. Neglecting its own weight, what load at the center will break this beam? Ans. 100,000 lb.

3. How thick should a bluestone sidewalk be to carry a total load of 300 pounds per square foot with a factor of safety of 10? The stones are supported at the curb and the building line by dwarf walls located 16 feet apart. Ans. 16 in.

4. A glass vault light is 6 inches long and is supported at the ends. It must carry 100 pounds per square inch with a factor of safety of 10. How thick should it be? Ans. 3 in.

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4. Concrete Beams.—The modulus of rupture of concrete is usually less than that of stone. One should think that this modulus of rupture would have about the same value as the ultimate unit tensile strength of the material since a plain-concrete beam breaks on the tension side. This, however, is not the case, and the modulus of rupture of concrete is about twice the ultimate unit tensile strength. Nevertheless, concrete is still very weak in withstanding bending stresses. For this reason, it is seldom used unreinforced for beams. Few tests that give reliable data have been made on the material so stressed. Another difficulty in regard to determining a value for the modulus of rupture of concrete is that many factors have a decided effect on this quantity. It is affected by the richness of the mixture, the amount of water used in making the concrete, its age, the care taken in mixing, the character of the aggregates, etc.

Table II gives values found by the United States Geological Survey at the government testing plant in St. Louis, Missouri. These tests are probably as complete as have ever been made on plain concrete. Three degrees of wetness are recognized in mixing the concrete, namely, wet, medium, and damp. *Wet concrete* is such that sufficient

TABLE II

MODULUS OF RUPTURE OF CONCRETE

Material	Mixture	Consistency	Modulus of Rupture at Various Ages					
			1 Day	1 Week	4 Weeks	13 Weeks	26 Weeks	1 Year
Neat cement		Water, 21 per cent. weight of dry materials	761	1,512	1,674	1,953	2,023	
Mortar	1–3	Water, 11.5 per cent. weight of dry materials		588	888	1,026	1,008	1,080
Mortar	1–4	Water, 11 per cent. weight of dry materials		390	636	852	864	870
Cinder concrete .	1–2–5	Wet			175	240	246	
Cinder concrete . .	1–2–5	Medium			198	231	277	
Cinder concrete . .	1–2–5	Damp			198	225	250	
Granite concrete . .	1–2–4	Wet			375	501	539	
Granite concrete . .	1–2–4	Medium			475	536	566	
Granite concrete . .	1–2–4	Damp			499	591	618	
Gravel concrete . .	1–2–4	Wet			391	380	435	
Gravel concrete . .	1–2–4	Medium			451	477	520	
Gravel concrete . .	1–2–4	Damp			426	495	496	
Limestone concrete .	1–2–4	Wet			422	487	507	
Limestone concrete .	1–2–4	Medium			458	541	566	
Limestone concrete .	1–2–4	Damp			537	521	589	

water is added to make it semiliquid; *damp concrete* is decidedly granular, with little tendency to lump; while *medium concrete* is between the other two mixtures. Table II is mostly for 1–2–4 mixtures. A 1–3–6 mixture, which is seldom used, gives values lower than the 1–2–4 mixture. The factor of safety employed is sometimes 6, but usually 10 or higher, as the strength of concrete is uncertain. Instead of using the modulus of rupture for stone and concrete, divided by a factor of safety, sometimes the ultimate tensile strength, properly reduced, is used, as the beam really fails by tension. Either method should lead to the same result, a safe unit bending stress. The best way is to select from experience this stress in the first place. Building laws usually specify working stresses. Many engineers state that plain concrete beams should never be used at all.

EXAMPLE.—Design a concrete beam 6 inches wide on a 6-foot span to carry at its center 800 pounds, which includes the equivalence of its own weight. The safe working stress is to be 30 pounds per square inch.

SOLUTION.—The moment is equal to $\frac{W l}{4} = \frac{800 \times 6 \times 12}{4} = 14{,}400$ in.-lb. Substituting the correct values, $14{,}400 = \frac{6 \times d^2}{6} \times 30$. Therefore, $d^2 = 28{,}800 \div 100 = 480$ and $d = 21.91$, say 22, in. Ans.

EXAMPLE FOR PRACTICE

1. A limestone concrete beam is made of a 1–2–4 medium mixture. It is 10 inches deep and 8 inches broad. What concentrated load at its center will break it at the end of 26 weeks, provided the span is 7 feet 6 inches and its own weight is neglected? Ans. 3,354 lb.

CONCRETE COLUMNS

5. Plain-concrete and stone columns may be divided into two classes, namely, those which are *centrally loaded* and those which are *eccentrically loaded.* In either case, the height of the column should never be more than twelve times the least dimension of the cross-section.

6. Column Centrally Loaded.—For a **centrally loaded column,** the allowable compressive stress per square inch is multiplied by the area of the cross-section of the column to find the allowable load. Thus, if it is decided to allow an intensity of stress of 300 pounds per square inch, and the column is of square section 10 inches on a side, the allowable load will be 10 × 10 × 300 = 30,000 pounds. The breaking load on columns between two and twelve times as high as the least dimension of their cross-section seems to be independent of their height. A column between these two limits, however, cannot withstand as high an intensity of stress as a cube, for it is more likely to break by shearing. For this reason, when employing values taken from Tables III, IV, and V for column calculations, a larger factor of safety should be used than with other concrete work. This factor is at least 6, and for stone and brick it is 10 or 15 or higher.

TABLE III

ULTIMATE UNIT CRUSHING STRENGTH OF STONE CONCRETE WITH PORTLAND-CEMENT MORTAR

Proportion of Ingredients			Compression Pounds per Square Inch			
Cement	Sand	Stone	7 Days	1 Month	3 Months	6 Months
1	2.0	4	1,600	2,150	2,400	2,500
1	2.5	5	1,430	1,950	2,250	2,350
1	3.0	6	1,250	1,800	2,100	2,200
1	3.5	7	1,100	1,660	1,960	2,080
1	4.0	8	980	1,520	1,820	1,950
1	4.5	9	850	1,400	1,690	1,840
1	5.0	10	750	1,260	1,550	1,720
1	5.5	11	650	1,120	1,420	1,600
1	6.0	12	600	1,000	1,300	1,500

Note.—For gravel concrete, use 75 per cent. of the figures given in the table; for cinder concrete, use 65 per cent.

Example 1.—What is the allowable working load on a concrete column that is 10 feet high and 12 inches in diameter and made of 1-2-4 stone concrete 6 months old with a factor of safety of 6?

Solution.—The cross-sectional area of the column is $.7854 \times 12^2 = 113.1$ sq. in. From Table III, the ultimate crushing strength of

1-2-4 concrete 6 mo. old is 2,500 lb. per sq. in. Using a factor of safety of 6, the safe intensity of stress is $\frac{2,500}{6}$. Then the safe total load the column can carry is

$$113.1 \times \frac{2,500}{6} = 47,125 \text{ lb.} \quad \text{Ans.}$$

TABLE IV

ULTIMATE UNIT CRUSHING STRENGTH OF STONES AND STONE MASONRY

Material	Compressive Strength Pounds per Square Inch	Material	Compressive Strength Pounds per Square Inch
Granite, Colorado	15,000	Limestone, Marquette, Michigan	8,000
Granite, Connecticut	14,000	Limestone, Conshohocken, Pennsylvania	15,000
Granite, Massachusetts	16,000	Marble, Montgomery Co., Pennsylvania	11,000
Granite, Maine	15,000	Marble, Lee (dolomite), Massachusetts	22,800
Granite, Minnesota	25,000	Marble, Pleasantville (dolomite), New York	22,000
Granite, New York	16,000	Marble, Italian	12,000
Granite, New Hampshire	12,000	Marble, Vermont	10,000
Bluestone	15,000	Slate	10,000
Sandstone, Middletown, Connecticut	7,000	Bluestone, ashlar piers	2,100
Sandstone, Longmeadow, Massachusetts	10,000	Granite, ashlar piers	2,100
Sandstone, Hudson River, New York	12,000	Limestone, ashlar piers	1,500
Sandstone, Little Falls (brown), New York	10,000	Common sandstone, ashlar piers	1,050
Sandstone, Ohio	8,000	Rubble piers, cement mortar	900
Sandstone, Hummelstown (brown), Pennsylvania	12,000	Rubble piers, lime mortar	480
Limestone, Kingston, New York	12,000		
Limestone, Garrison Station, New York	18,000		
Limestone, Bedford (oolitic), Indiana	8,000		

TABLE V

ULTIMATE CRUSHING STRENGTH OF BRICK MASONRY PIERS

(Average Age of Brickwork, 6 Months)

Material	Composition of Mortar	Compressive Strength Pounds Per Square Inch
Wire-cut brick	1 cement, 5 sand	3,000
Dry-pressed brick . .	1 cement, 5 sand	3,400
Dry-pressed brick . .	1 cement, 1 lime, 3 sand	2,300
Repressed brick . .	1 cement, 5 sand	1,700
Light-hard, sand-struck brick	1 cement, 5 sand	1,900
Light-hard, sand-struck brick	1 cement, 7 sand	853
Hard, sand-struck brick	1 cement, 1 sand	2,100
Hard, sand-struck brick	1 cement, 1 lime, 3 sand	1,500
Hard, sand-struck brick	1 cement, 5 sand	1,200
Sand-lime brick . .	1 cement, 3 sand	1,100
Sand-lime brick . . .	1 lime, 3 sand	450
Sand-lime brick . .	Neat cement	1,400
Terra-cotta work . . .	1 cement, 3 sand	2,000

EXAMPLE 2.—Design a square bluestone column, 12 feet high, to carry 1,000,000 pounds. Use a factor of safety of 10.

SOLUTION.—The allowable unit stress, according to Table IV, is 15,000 ÷ 10 = 1,500 lb. Therefore, the required area of the column is 1,000,000 ÷ 1,500 = 667 sq. in., and the edge of the column is $\sqrt{667}$ = 26 in. It will be noted that the load that this column is to carry is enormous, while the dimensions of the column do not seem very large. It is therefore evident that the architect in designing columns for architectural effect usually errs on the side of safety. Ans.

In designing concrete columns, a layer of concrete 1½ inches thick is sometimes added to the outside of the column to act as a fireproof covering. This precaution should always be taken where there is any danger of fire.

7. Eccentrically Loaded Column.—The stress on an **eccentrically loaded column** is computed by the following formulas where the eccentricity of the load is not great enough to cause tension in the column and is in rectangular columns normal to one face.

For circular columns,

$$s = \frac{P}{A} + \frac{8 e P_e}{A d} \qquad (1)$$

For rectangular columns,

$$s = \frac{P}{A} + \frac{6 e P_e}{A d} \qquad (2)$$

In these formulas,

s = stress, in pounds per square inch, developed in the column;
P = total load on the column, in pounds;
A = area of column section, in square inches;
e = eccentricity of eccentric part of load, in inches;
P_e = eccentric part of load, in pounds;
d = diameter of column, or dimension measured in the plane of the eccentricity, in inches.

If the total load is eccentric, the formulas just given reduce to the following:

For circular columns,

$$s = \frac{P_e}{A}\left(1 + \frac{8e}{d}\right) \qquad (3)$$

For rectangular columns,

$$s = \frac{P_e}{A}\left(1 + \frac{6e}{d}\right) \qquad (4)$$

According to these formulas, it is necessary first, in designing a column that will stand a given load, to select by inspection the section of column that seems to be about correct, and then to solve the equation for s. This value of s must be less than the allowable working stress it is proposed to use. If it is larger, a larger area of column must be selected, and the problem worked out again; if it is very much smaller, possibly too large a section for economy has been selected, and in this case a smaller section should be assumed and the equation again solved for s. It should be borne in mind that the height of the column must not be more than twelve times the least dimension of the cross-section, and under certain conditions not more than six times.

EXAMPLE 1.—Design a cylindrical column of 1-2-4 stone concrete, 18 feet high, to carry a central load of 100,000 pounds and an eccentric load of 100,000 pounds, the eccentricity being 4 inches.

SOLUTION.—Since the column is 18 ft. high, it should be at least 12 in. in diameter. The ultimate crushing strength may be taken as 2,500 lb. per sq. in. in 6 mo. The safe working stress would, therefore, be 2,500 ÷ 6 = 417 lb. per sq. in.

A column 28 in. in diameter will be tried first. The area of the cross-section is $.7854 \times 28^2$, or 615.75 sq. in. To apply formula **1**, $A = 615.75$, $P = 200{,}000$, $P_e = 100{,}000$, $e = 4$, and $d = 28$. Substituting in the formula,

$$s = \frac{200{,}000}{615.75} + \frac{8 \times 4 \times 100{,}000}{615.75 \times 28} = 325 + 186 = 511 \text{ lb. per sq. in.}$$

This stress is larger than the allowable stress, which shows that the column section selected is too small. If a section 31 in. in diameter is assumed, then,

$$A = .7854 \times 31^2 = 754.77 \text{ sq. in.}$$

Substituting in the formula,

$$s = \frac{200{,}000}{754.77} + \frac{8 \times 4 \times 100{,}000}{754.77 \times 31} = 402 \text{ lb. per sq. in.}$$

Since this is less than 417 lb., a column of this diameter is safe. Ans.

EXAMPLE 2.—What should be the size of a column of square section, to carry the same load as in example 1, the other conditions being the same?

SOLUTION.—Assuming a column 26 in. square, $A = 26^2 = 676$. Substituting in formula **2**,

$$s = \frac{200{,}000}{676} + \frac{6 \times 4 \times 100{,}000}{676 \times 26} = 433 \text{ lb. per sq. in.}$$

This is larger than 417, therefore a larger section, say 27 in. square, will be tried. $A = 27^2 = 729$. Substituting in the formula,

$$s = \frac{200{,}000}{729} + \frac{6 \times 4 \times 100{,}000}{729 \times 27} = 396 \text{ lb. per sq. in.}$$

Therefore, a column 27 in. square is sufficient. Ans.

EXAMPLES FOR PRACTICE

1. A column 15 inches in diameter and 15 feet high is made of 1–2–4 stone concrete; it carries a central load of 40,000 pounds and an eccentric load of 30,000 pounds. The eccentricity of the eccentric load being 3 inches, what is the intensity of stress produced?
Ans. 668 lb. per sq. in.

2. Design a round Maine granite column 18 feet high to carry 400,000 pounds centrally placed. Use a factor of safety of 10.
Ans. 18.426, say $18\frac{1}{2}$, in. diam.

REINFORCED CONCRETE

BEAMS

INTRODUCTION

8. Authorities Followed.—The design of reinforced concrete is not an exact science. Many engineers and manufacturers have devised excellent formulas to be used in design, and such formulas are usually named after their originator, as Thacher's formulas, Christophe's formulas, etc. It is proposed in this Section to follow somewhat closely the recommendations and formulas embodied in the excellent progress report of the "Joint Committee." This committee was composed of members of the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering and Maintenance-of-Way Association, and the Association of American Portland-Cement Manufacturers, "for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete."

9. Principal Considerations of Concrete Design. Concrete is a material of low tensile strength. Its valuable properties are durability, fire-resisting qualities, high compressive strength, and relatively low cost. Its strength increases with age. In the design of reinforced concrete, the tensile strength of the concrete is usually neglected and the steel rods that are inserted into the mass are assumed to take all the tension. Plain concrete, also, is seldom calculated to carry any tensile strains.

Failures of reinforced-concrete structures are usually due to three causes; namely, bad design, poor materials, and faulty workmanship. Of bad design, little may be said except that the utmost care must be exercised. Every detail of a structure must be carefully considered, as it is often on the careful and safe design of these details that the strength of a structure depends. Special attention must be paid to the connections of the various members of a structure. The unit stresses employed may be too high. No unit stresses to be used for all cases that may arise are recommended in this Section. The engineer must consult the building laws of the municipality in which the building is to be erected. These values are to be reduced, if necessary, according to the experience and judgment of the designer and according to the amount of care used in construction. The unit stresses employed in the examples in this Section are mostly those recommended by the Joint Committee. They are for concrete, mixed in the proportions to be specified, "capable of developing an average compressive strength of 2,000 pounds per square inch at 28 days, when tested in cylinders 8 inches in diameter and 16 inches long, under laboratory conditions of manufacture and storage, using the same consistency as is used in the field." These stresses are higher than are allowed in some large cities; but, again, the stresses allowed in some cities are considered by many engineers to be too conservative.

Only Portland cement should be used for reinforced concrete. Cinder concrete is not suitable, and only suitable first-class steel should be employed. The concrete should be properly made out of suitable and sufficient materials. It should be properly placed under suitable weather conditions, and the forms should be left up a sufficient length of time to allow it to attain sufficient strength to support itself.

10. Aggregates.—Concrete, as mentioned elsewhere, is made of cement and **aggregates.** The Joint Committee divides the aggregates into two divisions; namely, *fine* and *coarse.* "**Fine aggregate** consists of sand, crushed stone, or gravel screenings, passing, when dry, a screen having ¼-inch diameter holes. * * * **Coarse aggregate** consists of inert material, such as crushed stone or gravel, which is retained on a screen having ¼-inch diameter holes. For reinforced-concrete members, a size to pass a 1-inch ring, or a smaller size, may be used." In both fine and coarse aggregates, a gradation of the size of the particles is generally advantageous. In all cases, the aggregates should be of first-class quality, free from soft material and long or flat particles.

In regard to the proportions of aggregates, the Joint Committee states: "For reinforced-concrete construction, a density proportion based on 1–6 should generally be used; that is, 1 part of cement to a total of 6 parts of fine and coarse aggregates measured separately." If twice as much coarse aggregate as fine aggregate is used, this proportion will become 1–2–4, which is a richness of mixture often used for reinforced concrete. For important work, different proportions of fine and coarse aggregates may be made into test pieces of concrete, so that the strength of the concrete can be determined. In columns, a mixture of concrete richer than 1–2–4 is often required. In great masses, where the concrete is used principally for its compressive strength and its weight, poorer mixtures may be employed. In all cases, careful judgment should be exercised in selecting the proportions best suited to the conditions of the problem.

The concrete should be mixed with just enough clean water to permit it to flow into the forms and about the metal reinforcement and yet not cause the broken stone to separate from the cement and sand when the concrete is being conveyed from the mixer. Retempered concrete should not be used.

11. Details of Design.—In regard to stopping off work, the Joint Committee states: "For concrete construction, it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should, in general, be made at or near the center of the span. Joints in columns should be perpendicular to the axis of the column, and in girders, beams, and floor slabs perpendicular to the plane of their surfaces." Tongued and grooved expansion joints in plain concrete should be made about every 50 feet.

After concrete has been deposited in column forms it shrinks somewhat. For this reason, girders should not be constructed over columns until at least 2 hours after the columns are placed. If the concrete in the columns has become hard, it should be cleaned and slushed with mortar to insure a good bond.

12. One important factor to be considered in reinforced concrete is the *loads*. In the design of reinforced-concrete beams, columns, and slabs, the weight of the column, beam, or slab must never be neglected, because the dead load in reinforced concrete forms a large proportion of the total load. Therefore, the weight of the beam itself must always be included in the dead load. If a live load is suddenly applied, or is moving or vibrating so that the structure is subject to impact, vibration, or shock, special precautions must be taken. It is customary to add a certain percentage to such a load and thus reduce it to a quiescent live load that has an equivalent effect. Fortunately, such loads are rarely encountered except in railway bridges, which are problems for the bridge engineer.

The Joint Committee states that "the span length for beams and slabs shall be taken as the distance from center to center of supports, but shall not be taken to exceed the clear span plus the depth of the beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended."

DESIGN OF SIMPLE RECTANGULAR BEAMS

13. Neutral Axis.—In speaking of beams, the *neutral surface* is a surface above which, in a simple beam, the fibers of which the beam is composed are in compression and below which the fibers are in tension. If a cross-section of the beam is taken, this neutral surface will show as a line, and it is known as the **neutral axis,** as explained in *Theory of Beams*. Now, in ordinary beams made of a single material, as wood or steel or the like, this neutral axis always passes through the center of gravity of the section. With beams composed of concrete and steel, however, this is not the case. Here, the neutral axis is the line where there is no stress, but the center of gravity of the section is not located on it.

14. Assumptions Used in the Derivation of Formulas.—Two assumptions are made in deriving the formulas for beam design. The *first assumption* is that *all the tension is taken up by the steel.* Inasmuch as the concrete, as explained in Art. **9,** cracks easily, no reliance is put on its strength, and the steel rods are assumed to take all the tension. Above the neutral axis, however, the concrete withstands the compressive strains.

The *second assumption* is as follows: *Any plane cross-section of a reinforced-concrete beam remains a plane after the beam is loaded and, therefore, bent.* This matter will be better understood on referring to Fig. 2. In (*a*) is shown an end view of a beam, and in (*b*) a side view. The reinforcement is indicated by *a*. When the beam is loaded, it sags, or deflects, somewhat, and although this sag is usually so small that it cannot be seen, it is there nevertheless. This sag is due to the fact that the fibers in the top of the beam compress and shorten up while the steel in the bottom of the beam stretches.

In (*c*), a portion of the beam is shown on an enlarged scale. This portion of the beam is the one just at the right of the cutting plane *e f* in view (*b*). When the beam is loaded, the fiber at *g* is compressed and pushed over to *g'*; also, the steel at *h* is stretched until it extends to *h'*. Now, *g* and *h* are in the cutting plane and, according to the second assumption, *g'* and *h'*, which are the new positions of the fibers, are also in one plane. Of course, the distances *g g'* and *h h'* are greatly exaggerated in the figure to make this matter clear.

15. Notation Used.—In the formulas about to be considered, the following notation will be used:

s_s = tensile stress, per square inch, in steel;
s_c = maximum compressive stress, per square inch, in concrete;
E_s = modulus of elasticity of steel;
E_c = modulus of elasticity of concrete;
n = ratio of modulus of elasticity of steel to that of concrete $= \frac{E_s}{E_c}$;
M = resisting moment or bending moment, in inch-pounds;
A = area of steel in tension, in square inches;
b = breadth of beam, in inches (see Fig. 2);
d = depth of beam, in inches, from center of steel to top of beam (see Fig. 2);
p = proportion of steel $= \frac{A}{b\,d}$.

Both k and j are coefficients to be regarded as follows: Let the distance from the neutral surface to the top of the beam be called x. Then, the value of k is taken so that $k = \frac{x}{d}$. Then, $x = k\,d$, which is the value given in Fig. 2 (*b*). In what follows, it is always more convenient to use $k\,d$ in place of x. In a similar manner, the distance from the center of the steel to the center of compression of the concrete is called $j\,d$, as shown in (*b*).

16. Derivation of Formulas.—Consider Fig. 2 (*c*). The angle $h\,N\,h'$ is equal to the angle $g\,N\,g'$. Both the angle $N\,g\,g'$ and the angle $N\,h\,h'$ are right angles; therefore, the triangles $g\,g'\,N$ and $h\,h'\,N$ are similar and

$$\frac{\text{length } h\,h'}{\text{length } g\,g'}=\frac{\text{length } N\,h}{\text{length } N\,g}$$

Now, the length $N\,g=k\,d$ and the length $N\,h=d-k\,d$; also, $g\,g'$ is the deformation in the fiber of concrete at the top of the beam. The modulus of elasticity is equal to the stress divided by the deformation, or strain; that is, $E_c=\frac{s_c}{g\,g'}$, or $g\,g'=\frac{s_c}{E_c}$. Likewise, $h\,h'=\frac{s_s}{E_s}$. Putting these values in the equation just given,

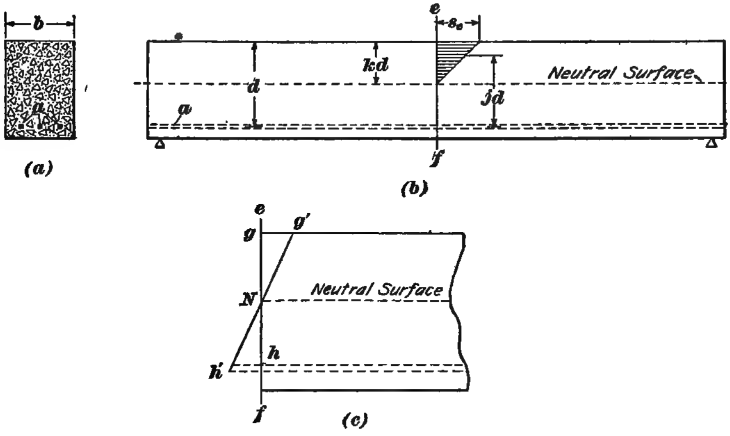

FIG. 2

$$\frac{s_s\,E_c}{s_c\,E_s}=\frac{d-k\,d}{k\,d}$$

However, $\frac{E_s}{E_c}=n$, or $\frac{E_c}{E_s}=\frac{1}{n}$. Substituting this value,

$$\frac{s_s}{s_c\,n}=\frac{d-k\,d}{k\,d}=\frac{d(1-k)}{d\,k}=\frac{1-k}{k}$$

Next, consider the section *e f* in Fig. 2 (*b*). The concrete above the neutral axis is pressing apart the two parts of the beam on both sides of this section, and the steel below the neutral axis is holding together the two parts of the beam. It is evident that the concrete cannot push harder than the steel can pull, for if it did the two halves of the beam would separate. Therefore, the pull in the steel must just equal the push in the concrete. The amount of pull in the steel is, of course, equal to the area of the steel multiplied by the unit stress in the steel, or $A\,s_s$. The total compression in the concrete can be seen by examining view (*b*). The greatest unit stress in the concrete is s_c. This stress occurs at the top surface of the beam, and from this point down the stress decreases until at the neutral axis it is zero. The shaded triangle in this view will serve to give an idea of the change of stress. The area on which the compression acts is $b \times k\,d$, and the *average* stress acting on this area is one-half the maximum stress, or $\frac{s_c}{2}$. The total compression is therefore $\frac{b\,k\,d\,s_c}{2}$. Equating this to the tension,

$$A\,s_s = \frac{b\,k\,d\,s_c}{2}$$

Transposing,

$$\frac{s_s}{s_c} = \frac{b\,k\,d}{2\,A}$$

By transposing n in the equation $\frac{s_s}{s_c\,n} = \frac{1-k}{k}$, which has been already derived, there results

$$\frac{s_s}{s_c} = \frac{n\,(1-k)}{k}$$

Equating these two values of $\frac{s_s}{s_c}$,

$$\frac{b\,k\,d}{2\,A} = \frac{n\,(1-k)}{k}$$

But $\frac{A}{b\,d} = p$, or $\frac{b\,d}{A} = \frac{1}{p}$. Substituting this value,

$$\frac{k}{2\,p}=\frac{n\,(1-k)}{k}$$

and solving for k,

$$k=\sqrt{2\,p\,n+(p\,n)^2}-p\,n \qquad (1)$$

The value for j will next be found. The center of compression of the concrete is, of course, at the center of gravity of the shaded triangle which represents the stress in the concrete; that is, one-third of $k\,d$ from the top of the beam. The value of j is therefore found as follows:

$$j\,d=d-\tfrac{1}{3}\,k\,d=d\,(1-\tfrac{1}{3}\,k)$$

Then, $$j=1-\tfrac{1}{3}\,k \qquad (2)$$

The resisting moment of the beam must now be found. This is equal to the total compression of the concrete multiplied by the distance of the center of pressure from the neutral axis, added to the total tension in the steel multiplied by the distance of its center of tension from the neutral axis. As the total compression in the concrete is equal to the total tension in the steel, the resisting moment is equal to either the total tension in the steel or the total compression in the concrete multiplied by the distance between the center of tension and the center of compression, or $j\,d$. Therefore, the resisting moment may be expressed in terms of either the compression in the concrete or the tension in the steel. Therefore,

$$M=A\,s_s\,j\,d=p\,s_s\,j\,b\,d^2$$

and $$M=b\,k\,d,\,\frac{s_c}{2}\,j\,d$$

Transposing both these equations so as to solve for the stress,

$$s_s=\frac{M}{A\,j\,d}=\frac{M}{p\,j\,b\,d^2} \qquad (3)$$

and $$s_c=\frac{2\,M}{j\,k\,b\,d^2} \qquad (4)$$

17. To Design a Simple Beam.—To design a beam, first assume values for p, n, and either d or b, preferably the former, and find k by the formula $k=\sqrt{2pn+(pn)^2}-pn$. Then find j by the formula $j=1-\frac{1}{3}k$, and calculate M from the loads to be carried. Next, assume a value for s_s and find either b or d by the formula $s_s=\frac{M}{A\,j\,d}=\frac{M}{p\,j\,b\,d^2}$. Then find s_c by the formula $s_c=\frac{2M}{j\,k\,b\,d^2}=\frac{2\,p\,s_s}{k}$.

The value found for s_c must not be excessive. If it is, a new value must be assumed for p and the problem reworked or the moment reduced until s_c is safe.

18. As an example, design a beam on a 20-foot span to carry 1,000 pounds per foot. This load includes its own weight. Assume for the example that the total depth of the beam is 30 inches, and as the steel must be kept up a little from the bottom, let $d=28$ inches. Assume that $p=.006$, and as a value for n take 15. Then,

$$k=\sqrt{2\,p\,n+(p\,n)^2}-p\,n$$
$$=\sqrt{2\times.006\times15+(.006\times15)^2}-.006\times15=.343$$

and $$j=1-\tfrac{1}{3}k=1-\tfrac{1}{3}\times.343=.886$$

Assume that $s_s=16{,}000$ pounds and that $A=p\,b\,d=.006\,b\times28$. Then,

$$\frac{W\,l}{8}=\frac{(20\times1{,}000)\times20}{8}=50{,}000 \text{ foot-pounds}$$

Therefore, $M=50{,}000\times12=600{,}000$ inch-pounds.

Substituting these values in the formula $s_s=\frac{M}{A\,j\,d}$, it becomes $16{,}000=\frac{600{,}000}{.006\,b\times28\times.886\times28}$, which reduces to $b=8.998$, say 9, inches.

Substituting the correct values in the formula $s_c=\frac{2M}{j\,k\,b\,d^2}$,

$$s_c=\frac{2\times600{,}000}{.886\times.343\times9\times28\times28}=559.6, \text{ say } 560,$$

pounds per square inch.

If the designer considers this value of s_c to be safe, the beam may be said to be correctly designed.

19. The objection to the method of design just given is that s_s is assumed and s_c is found. This value of s_c may be too large or too small, and perhaps a new value of p must be assumed and the calculations remade. Of course, a beam is most economical when the maximum allowable unit stress in the steel is realized under the same load that realizes the maximum allowable unit stress in the concrete. If the steel reaches its maximum allowable stress first, then the beam cannot be loaded further until the maximum allowable unit stress in the concrete is reached, because the steel would then be overstressed. In such a condition, not enough steel is used to develop the full stress in the concrete. On the other hand, if too much steel were used, the strength of the beam would be limited not by the maximum stress in the steel, but by the maximum stress in the concrete which would be developed under a smaller load. Often, in practical design, conditions are imposed that fix the value of p or d, when the beam may be designed as just outlined. Its resisting moment is in this case limited either by the stress in the concrete or that in the steel. However, when nothing else interferes, it is often convenient to select such a value of p that the maximum allowable stresses in the concrete and steel are both reached under the same load. Some engineers call such a value of p the *most economical value* and others refer to it as the *critical value.*

To find this amount of steel, the following method is adopted: From the discussion in Art. **16,**

$$\frac{s_s}{s_c n} = \frac{1-k}{k}$$

Solving for k, it becomes

$$k = \frac{n\frac{s_c}{s_s}}{1+n\frac{s_c}{s_s}}$$

Since, at any section, the total compression in the concrete is equal to the total tension in the steel,

$$\frac{s_c\,k\,d\,b}{2}=A\,s_s$$

Transposing, $$k=\frac{2\,A\,s_s}{b\,d\,s_c}$$

But $\frac{A}{b\,d}=p$; therefore,

$$k=\frac{2\,p\,s_s}{s_c}$$

Equating the two values of k thus found,

$$\frac{n\frac{s_c}{s_s}}{1+n\frac{s_c}{s_s}}=\frac{2\,p\,s_s}{s_c}$$

Solving for p, $$p=\tfrac{1}{2}\,\frac{1}{\frac{s_s}{s_c}\left(\frac{s_s}{n\,s_c}+1\right)}$$

20. A beam can now be designed by using the critical value for p. First assume values for s_s, s_c, and n and find p by the formula

$$p=\tfrac{1}{2}\,\frac{1}{\frac{s_s}{s_c}\left(\frac{s_s}{n\,s_c}+1\right)} \qquad (1)$$

Then find k by the formula

$$k=\sqrt{2\,p\,n+(p\,n)^2}-p\,n \qquad (2)$$

and find j by the formula

$$j=1-\tfrac{1}{3}\,k \qquad (3)$$

Next, assume either d or b and find the other by either the formula

$$s_s=\frac{M}{A\,j\,d}=\frac{M}{p\,j\,b\,d^2} \qquad (4)$$

or the formula

$$s_c = \frac{2\,M}{j\,k\,b\,d^2} \qquad \textbf{(5)}$$

If a value for p other than that found by formula **1** is used, then both formulas **4** and **5** must be solved so that the safe value of neither s_s nor s_c will be exceeded.

In actual practice, the designer fixes on values for n, s_s, and s_c and solves formulas **1**, **2**, and **3** once for all. The values assumed in this Section will be $n = 15$, $s_s = 16{,}000$, and $s_c = 650$ for concrete capable of developing a compressive stress of 2,000 pounds in 28 days, tested as previously stated and for steel specified by the committee. Many cities do not allow values so high, and the designer must be governed by the law. However, as some values must be used for illustration, the ones just given will be used in this Section to serve as examples and because they are recommended by the Joint Committee. It is therefore proposed to solve formulas **1**, **2**, and **3** once and for all for this Section.

Using values recommended by the Joint Committee and substituting them in the formula $p = \frac{1}{2} \times \frac{1}{\frac{s_s}{s_c}\left(\frac{s_s}{n\,s_c} + 1\right)}$,

$$p = \tfrac{1}{2} \times \frac{1}{\frac{16{,}000}{650}\left(\frac{16{,}000}{15 \times 650} + 1\right)} = .00769$$

Substituting this value in formula **2**, or

$$k = \sqrt{2\,p\,n + (p\,n)^2} - p\,n,$$

$$k = \sqrt{2 \times .00769 \times 15 + (.00769 \times 15)^2} - .00769 \times 15 = .379$$

Substituting this value in formula **3**, or $j = 1 - \frac{1}{3}\,k$,

$$j = 1 - \tfrac{1}{3} \times 379 = .874$$

If s_s is taken at 15,000 to 16,000 and s_c at 600 to 650, the Joint Committee states that for approximate results j may be taken at $\frac{7}{8}$. This would make k equal to $\frac{3}{8}$.

21. As an example, design a concrete beam to carry 900 pounds per foot on a span of 30 feet. This load includes the weight of the beam. The total bending moment is $\frac{900\times 30\times 30}{8}=101{,}250$ foot-pounds, or $101{,}250\times 12=1{,}215{,}000$ inch-pounds. Taking the approximate values $k=\frac{3}{8}$ and $j=\frac{7}{8}$, assume that $d=26$ inches. Substituting the correct values in formula **5**, Art. **20**, $650=\frac{2\times 1{,}215{,}000}{\frac{7}{8}\times\frac{3}{8}\times b\times 26^2}$; therefore,

$$b=\frac{2\times 1{,}215{,}000}{\frac{7}{8}\times\frac{3}{8}\times 650\times 26^2}=16.85\text{, say }16\tfrac{7}{8}\text{, inches}$$

The area of steel required is, then, $p\,b\,d=.00769\times 16\frac{7}{8}\times 26=3.374$ square inches. As the economical value for p was used, the work can be checked by formula **4**, Art. **20**; thus,

$$s_s=\frac{M}{A\,j\,d}=\frac{1{,}215{,}000}{3.374\times\frac{7}{8}\times 26}=15{,}829\text{ pounds,}$$

which is a close approximation to 16,000 pounds.

If the value of p is not the critical value, but is near it, the values found for j and k may be used for approximate results, because when used in their formulas under proper conditions both err on the side of safety.

EXAMPLE.—Design a beam on a 27-foot span to carry besides its own weight a load of 3,700 pounds per foot.

SOLUTION.—First, a value for the depth of the beam must be assumed. Thus, assume that the beam is 50 in. deep from the top surface to the bottom surface. As the steel must be up a few inches in the concrete, let it be assumed that $d=48$ in. The weight of the beam is not known and must be assumed. It may be assumed at 1,100 lb. per ft. After the design is completed, the accuracy of this assumption may be tested.

The total load is then $(3{,}700+1{,}100)\times 27=129{,}600$ lb. The bending moment is $\frac{129{,}600\times 27\times 12}{8}=5{,}248{,}800$ in.-lb. For approximate results, assume, as stated by the Joint Committee, that $j=\frac{7}{8}$ and $k=\frac{3}{8}$. Substituting the correct values in formula **5**, Art. **20**, it becomes

$$s_c=\frac{2\,M}{j\,k\,b\,d^2}=650=\frac{2\times 5{,}248{,}800}{\frac{7}{8}\times\frac{3}{8}\times b\times 48^2}$$

Therefore, $b=21.36$, say $21\frac{3}{8}$, inches. $A=p\,b\,d=.00769\times 21\frac{3}{8}\times 48=7.89$ sq. in.

Checking by formula **4**, Art. **20**,

$$s_s=\frac{M}{A\,j\,d}=\frac{5{,}248{,}800}{7.89\times\frac{7}{8}\times 48}=15{,}839,$$

which is a fairly close approximation.

The weight per foot of the beam is $\frac{50\times 21\frac{3}{8}\times 150}{144}=1{,}113$ lb. This is 13 lb. more than the assumed weight, but for many purposes it is close enough. The problem, therefore, need not be reworked. Ans.

Example 2.—Using the critical value of p, design a floor slab on a 12-foot span to carry safely 500 pounds per square foot besides its own weight.

Solution.—In the design of a slab it is customary to consider a section of the slab 12 in. wide. Assume first that the weight of the slab is 150 lb. The total load per square foot is then 650 lb. and the load on the entire strip is $650\times 12=7{,}800$ lb. The moment is therefore

$$\frac{W\,l}{8}=\frac{7{,}800\times 12\times 12}{8}=140{,}400 \text{ in.-lb.}$$

It is noted that a width of slab of 12 in. is considered. Therefore, the value of b is fixed and that of d is found by formula **5**, Art. **20**. Thus,

$$s_c=\frac{2\,M}{j\,k\,b\,d^2}=650=\frac{2\times 140{,}400}{\frac{7}{8}\times\frac{3}{8}\times 12\times d^2}$$

Therefore, $d=10.48$, say $10\frac{1}{2}$, in.

A, or the area of steel in a strip 12 in. wide, is $A=p\,d\,b=.00769\times 10\frac{1}{2}\times 12=.97$ sq. in. for each foot in width. Checking the design by formula **4**, Art. **20**,

$$s_s=\frac{M}{A\,j\,d}=\frac{140{,}400}{.97\times\frac{7}{8}\times 10\frac{1}{2}}=15{,}754 \text{ lb.},$$

which is a close approximation.

Now, below the steel there should be 1 in. of concrete; therefore, the total depth of the slab will be $10\frac{1}{2}+1=11\frac{1}{2}$ in. A square foot of slab therefore weighs $\frac{11\frac{1}{2}}{12}\times 150=144$ lb., which is close enough to 150, the value assumed. Ans.

EXAMPLES FOR PRACTICE

1. Design to the nearest half inch a concrete slab on a 14-foot span to carry 500 pounds per square foot, besides its own weight. Put 1 inch of concrete below the reinforcement.

$$\text{Ans.}\begin{cases}13\tfrac{1}{2}\text{ in. total thickness}\\ 1.15\text{ sq. in. of steel per ft.}\end{cases}$$

2. Design a beam to carry 1,000 pounds per foot on a 12-foot span. This load includes the weight of the beam. Make the total depth of the beam 17 inches and place the steel 2 inches from the bottom of the beam.

$$\text{Ans.}\begin{cases}\text{Width of beam, a little over 9 in., say } 9\tfrac{1}{4}\text{ in.}\\ \text{Area of steel, 1.07 sq. in.}\end{cases}$$

3. Design a beam to carry a total uniformly distributed load of 100,000 pounds on a 27-foot span. Make the total depth of the beam 55 inches and raise the steel up 2 inches from the bottom.

$$\text{Ans.}\begin{cases}\text{Width of beam, a little over } 13\tfrac{1}{2}\text{ in., say } 13\tfrac{3}{4}\text{ in.}\\ \text{Area of steel, 5.6 sq. in.}\end{cases}$$

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22. In all the preceding problems, k was taken as $\frac{3}{8}$ and j as $\frac{7}{8}$ in order to get uniform results. It must be remembered, however, that these values are approximate when using values for s_c, s_s, and n as already given or when using values near them. These approximate values will often be found close enough for practical work. Nevertheless, the designer must always bear in mind that the values are approximate, and if greater accuracy is required, the method explained in the first part of Art. **20** must be followed.

For various reasons the value of p is not always taken at .00769, but is sometimes taken near that value. If it is less than the critical value, then b may be found approximately by the formula $s_s = \frac{M}{p\,j\,b\,d^2}$.

In a similar manner, if the value of p is more than the critical value, the formula $s_s = \frac{M}{p\,j\,b\,d^2}$ will not give safe values, because it is the concrete that will then be overstressed first and b must be found by the formula $S_c = \frac{2\,M}{j\,k\,b\,d^2}$. If the value for p is not the critical value, but is near it, the values found for j and k may be used for approximate results, because when they are used in their proper places both err on the side of safety.

23. The problem of design is often presented in this way: The values of b and d are determined and the beam must be designed to withstand a given moment M. According to formula **4**, Art **20**, $s_s=\frac{M}{A\,j\,d}$. Substitute in this formula values for s_s, M, and d. The value of j must be assumed to be $\frac{7}{8}$; its exact value cannot be found because p is not known. Solve this formula for A. As a check, find p from $p=\frac{A}{b\,d}$. If the value thus found is less than the critical value, the beam is safe, because the value of j assumed is for the critical value and is greater for all values of p less than this.

24. As an example, find out how much steel is required in a beam to resist safely a bending moment of 200,000 inch-pounds if $b=12$ and $d=20$.

Here, $16{,}000=\frac{200{,}000}{A\times\frac{7}{8}\times 20}$. Therefore, $A=.7143$ square inch and $p=\frac{.7143}{12\times 20}=.00298$. As this value is less than the critical value, the beam will not fail on account of crushing the concrete, and the design is therefore safe. The assumed value of j is for a critical value of p; for the actual value of p, it is found to be too small. This inaccuracy, however, errs on the side of safety. If desired, a slightly smaller value of A may be assumed and the problem checked by finding a value for j that corresponds exactly to this value, but such refinement is not often followed.

EXAMPLES FOR PRACTICE

1. A beam is 10 inches wide and d is equal to 14 inches. It carries a total load of 1,000 pounds per foot, including its own weight, on a span of 8 feet. How much steel is required? Ans. .484 sq. in.

2. How much steel is required in a beam when $M = 500{,}000$ inch-pounds, $b = 16$ inches, and $d = 24$ inches. Ans. 1.488 sq. in.

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25. It is understood that the preceding method is safe only when the concrete is not overstressed. This is taken care of by using the formula only when p comes less than the critical value.

The Trussed Steel Concrete Company, manufacturers of the Kahn bars, publish a popular approximate form of the formula used in the preceding problems. It is derived as follows: First the value for s_c is taken at 750 pounds per square inch. This value is high, but is sometimes used for the best rock concrete. With s_s equal to 16,000, with the preceding value of s_c, and with n equal to 15, the value of p, from formula **1**, Art. **20**, is about .01 and that of j is about .86. Substituting these values in the formula $s_s = \frac{M}{A\,j\,d}$ and transposing, it becomes $M = .86 \times 16{,}000\,d\,A$, or $M = 13{,}760\,d\,A$. In other words, to design a beam, assume d and find M, and then solve for A by the formula $M = 13{,}760\,d\,A$. The value of A found must be less than 1 per cent. of $b\,d$. This formula assumes stresses in the concrete of 750 pounds, which is high; if lower stresses in the concrete are desired, the value of A should be kept less than 1 per cent of $b\,d$.

The same manufacturers use the same formula for **T**-shaped beams when the neutral axis comes in the slab. If the slab is sufficiently thick and sufficiently broad, 2 or more per cent. of steel may be used in **T** beams, meaning of course, 2 per cent. of the rectangular part of the beam above the steel. For safety, however, the stresses created should be checked by the methods given later or by the tables made for that purpose.

26. To Investigate a Simple Beam.—It often happens that a beam is already built, or at least the dimensions are decided on, and it is necessary to decide whether the beam will carry the required load. In such cases, b, d, and A are measured and p is calculated; also, M can be calculated from the load that the beam carries or is to carry. First, a value of n is assumed and k is found by the formula $k=\sqrt{2\,p\,n+(p\,n)^2}-p\,n$. Then j is found by the formula $j=1-\frac{1}{3}k$. Next, s_s is found by the formula $s_s=\frac{M}{A\,j\,d}$ and s_c is found by the formula $s_c=\frac{2\,M}{j\,k\,b\,d^2}$. If p is greater than the critical amount, the strength is limited by the stress in the concrete and $s_s=\frac{M}{A\,j\,d}$ need not be solved. If p is less than the critical amount, the strength is limited by the stress in the steel and the formula $s_c=\frac{2\,M}{j\,k\,b\,d^2}$ need not be solved.

27. As an example, assume that a certain beam is 12 inches wide and 24 inches deep. It is reinforced by two $1\frac{1}{2}$-inch round bars. This reinforcement is 2 inches from the bottom of the beam. It carries a load of 700 pounds per foot, which includes its own weight. The span is 24 feet. Is the beam safe?

The area of a $1\frac{1}{2}$-inch round rod is about $1\frac{3}{4}$ square inches. Therefore, the area of both rods is $3\frac{1}{2}$ square inches. Then,

$$p=\frac{A}{b\,d}=\frac{3.5}{12\times 22}=.0133$$

Assume, as elsewhere, that $n=15$. Therefore,

$$k=\sqrt{2\times .0133\times 15+(.0133\times 15)^2}-.0133\times 15=.4629$$

and

$$j=1-\tfrac{1}{3}\times .4629=.8457$$

Now, the value of p is greater than the critical amount. Therefore, the strength is limited by the stress in the concrete. The maximum moment is $\frac{700\times 24\times 24\times 12}{8}=604{,}800.$

Substituting the correct values, therefore, in the formula $s_c = \frac{2M}{j\,k\,b\,d^2}$,

$$s_c = \frac{2 \times 604{,}800}{.8457 \times .4629 \times 12 \times 22 \times 22} = 532 \text{ pounds,}$$

which is safe. As p is greater than the critical amount, s_s is still safer.

28. Sometimes the problem is varied somewhat from that already given, and is as follows: A beam of a certain size has a certain amount of steel. What safe load will it carry?

In such a case, the method of procedure is almost the same as before. First, find p, j and k, as usual; then assume s_s and s_c and find M by the formula $s_s = \frac{M}{A\,j\,d}$, or $s_c = \frac{2M}{j\,k\,b\,d^2}$. If p is less than the critical percentage, use the first formula; if more than the critical percentage, use the second one.

29. As an example, assume that a beam is 15 inches wide and has a total depth of 30 inches. The steel consists of three 1-inch square rods raised 2 inches off the bottom. What safe uniformly distributed load will the beam carry on a span of 10 feet?

First, find the value of p. In this case it is $\frac{3}{15 \times 28} = .00714$. Then,

$$k = \sqrt{2 \times .00714 \times 15 + (.00714 \times 15)^2} - .00714 \times 15 = .368$$

and
$$j = 1 - \tfrac{1}{3} \times .368 = .877$$

As p is less than the critical amount, the steel limits the strength of the beam because it will exceed its safe value first. Therefore,

$$s_s = \frac{M}{A\,j\,d}, \text{ or } 16{,}000 = \frac{M}{3 \times .877 \times 28}$$

Then, $M = 1{,}178{,}688$ inch-pounds. The moment is equal, of course, to $\frac{W\,l}{8}$; therefore, as $l = 10 \times 12 = 120$ inches, it is

$1,178,688 = \frac{W \times 120}{8}$, and $W = 78,579$ pounds. The load per foot is therefore $78,579 \div 10 = 7,858$ pounds. The beam itself weighs $\frac{15 \times 30}{144} \times 150 = 469$ pounds per foot. Therefore, the total extra load per foot that the beam will safely carry, besides its own weight, is $7,858 - 469 = 7,389$ pounds.

EXAMPLES FOR PRACTICE

1. A beam 7 inches in width and 16 inches in total depth is reinforced with one $1\frac{1}{4}$-inch round rod placed $1\frac{1}{2}$ inches from the bottom. What stress is produced in the concrete if it carries besides its own weight 2,740 pounds per foot on a span of 6 feet 8 inches?

Ans. $s_c = 680$ lb., which is high, but is sometimes used

2. A slab is 4 inches thick and it is reinforced with $\frac{3}{8}$-inch round rods placed 6 inches apart. The rods are up 1 inch from the under surface of the slab. The span is 6 feet. What load besides its own weight will it safely carry? Ans. About 124 lb. per sq. ft.

CONTINUOUS BEAMS

30. The beams so far considered have been simple beams. The bending moments have, of course, been calculated by the ordinary bending-moment formulas. Thus, for a uniformly distributed load, the moment is $\frac{W\,l}{8}$, in which W is the total load and l is the span from center to center of supports. If a beam or a slab is *continuous*, the bending moment between the supports is reduced and a negative moment is created over each support. It is not possible to calculate exactly how much the bending moment is reduced by making the beam continuous, and, besides, the matter is not looked upon in the same light by all engineers.

The Joint Committee recommends that for the inside spans of all continuous beams and floor slabs having more than three supports and of moderate length, the moment be taken as $\frac{W\,l}{12}$ instead of $\frac{W\,l}{8}$, and that the same moment be considered to act in the opposite direction over each support. The end spans of continuous beams are continuous only at one end, the outside end being of course like a simple beam. The Joint Committee recommends that for beams the bending moment at the middle of the end span be considered as $\frac{W\,l}{10}$, and the negative moment at the adjoining support be taken at a like amount. The bending-moment formulas, of course, are for a uniformly distributed load. No formulas are offered for concentrated loads, and the designer must use his own judgment when reducing moments in such cases. After the bending moment is found, the problem is solved as usual.

31. As an example, design a middle span of a continuous beam to carry a load, including its own weight, of 1,000 pounds per foot; the span is 12 feet.

The total load is 1,000×12=12,000 pounds, and the maximum moment is $\frac{W\,l}{12}=\frac{12{,}000\times 12}{12}=12{,}000$ foot-pounds, or 144,000 inch-pounds. Assume that $d=12$, and use the critical value of p, which is taken as .00769; then, substituting the values in the equation $s_c=\frac{2\,M}{j\,k\,b\,d^2}$, $650=\frac{2\times 144{,}000}{\frac{7}{8}\times\frac{3}{8}\times b\times 12^2}$.

Therefore, $b=9.38$, say $9\frac{1}{2}$, inches and $A=.00769\times 12+9\frac{1}{2}=.88$ square inch. Checking by $s_s=\frac{M}{A\,j\,d}$, it is $s_s=\frac{144{,}000}{.88\times\frac{7}{8}\times 12}=15{,}584$ pounds per square inch, which is close. This metal must be put at the center of the span, and over each support at the top of the beam.

Usually, part of the reinforcement in the bottom of the beam is bent upwards to form the reinforcement over the supports, and additional reinforcement must sometimes be added to make up the required amount. This reinforcement in the top should extend as far as there is any reverse bending moment, usually one-quarter of the span each way, or three-tenths. The rods bent up from the bottom may usually be bent up so that they will cross the neutral surface at the point of contraflexure. Care must be taken to insure that the rods at the top of the beam over the supports are properly anchored so as to prevent them from slipping out of the concrete.

Some engineers claim that the effect of the reverse bending moment is uncertain, owing to the liability of one support settling slightly. Building laws usually cover this practice. If there is any chance of live loads being unbalanced on adjacent spans, it is illogical to consider the beam as continuous so far as the live load is concerned. Whether the beams are designed as continuous or not, the reinforcement should be secured over the supports to take up reverse bending moments.

SLABS REINFORCED IN BOTH DIRECTIONS

32. Sometimes square slabs have been reinforced in both directions. The gain is more apparent than real. Although the bending moment each way is about half the original in square slabs, the exact moments and the method of failure are not known, which makes accurate design impossible. Also, the two extra side beams must be strengthened to carry the slab at the sides. The load on the four supporting beams is not uniform but is a maximum at the center and is assumed to reduce to zero at each beam support, a more severe condition than a uniform load. If the slab is not square, the shorter span carries by far the larger part of the load, and if it is 1½ or more times as long as wide it should never have cross reinforcement. Slabs supported on four sides are not recommended, as they cannot be accurately designed, especially those that are not exactly square.

BEAMS WITH DOUBLE REINFORCEMENT

33. Sometimes, when a beam is used in certain locations, as, for example, a lintel over a window, both its thickness and its width are limited. If there is not sufficient room to build a beam as ordinarily designed to carry the required load, the beam must be strengthened in some manner.

One way of strengthening the beam is to insert a steel

I beam in the concrete. In such a case, the **I** beam should be designed to carry all the load.

If it is decided to use true reinforced concrete, resort must be had to some other expedient. By increasing the amount of reinforcement in the bottom of the beam, the concrete in the top of the beam will fail first. It is true that increasing the amount of steel reinforcement will lower the position of the neutral axis. Therefore, if the amount of reinforcement is doubled, the beam will hold about 20 per cent. more load than when no excess of reinforcement occurs, although the concrete will fail first. An increase of strength gained by such a method is too expensive.

The usual way of increasing the strength of a beam, the size of which is limited, is by using **double reinforcement;** that is, reinforcement placed at both the top and the bottom of the beam. The reinforcement at the top is in compression and assists the surrounding concrete to withstand the stress put upon it. Although this method is not any too economical, it is the best one to follow when the necessity arises.

34. The derivation of the formulas for such reinforcement will be omitted here, and only the formulas themselves will be given. They are derived along lines similar to those used for simple beams. The same notation is used as with simple beams, but the following additional characters are used:

A' = area of compressive steel, in square inches;
p' = ratio of compressive steel $= \frac{A'}{b\,d}$;
s_s' = unit compressive stress in steel, in pounds per square inch;
d' = depth from center of compressive steel to top of beam, in inches;
z = distance from center of compression to top of beam, in inches.

35. The diagram of the conditions of the beam is shown in Fig. 3. In double-reinforced beams the breadth and the total depth are known from the conditions of the problem. The value of d is $1\frac{1}{2}$ or 2 inches less than the total depth. Then, virtually, b and d are known. Also, the bending moment is known. Then, from formula **3**, Art. **16**, an approximate value of A may be found. Subtract from this value of A the critical amount of reinforcement. The remainder is the steel that must be taken care of by steel in compression. Usually, it will take about $2\frac{1}{4}$ square inches of steel in compression for each square inch of the calculated excess of steel in tension. The compressive steel is usually so placed that d' equals $1\frac{1}{2}$ to 2 inches. After the beam is thus assumed, it must be investigated to see whether or not the stresses are within safe limits, as follows:

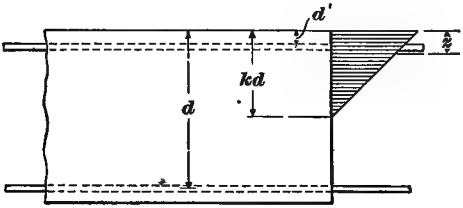


FIG. 3

Find k by the formula,

$$k=\sqrt{2\,n\left(p+p'\frac{d'}{d}\right)+n^2(p+p')^2}-n(p+p') \qquad (1)$$

Then find z by the formula,

$$z=\frac{\frac{1}{3}\,k^3 d+2\,p'\,n\,d'\left(k-\frac{d'}{d}\right)}{k^2+2\,p'n\left(k-\frac{d'}{d}\right)}, \qquad (2)$$

find $j\,d$ by the formula

$$j\,d=d-z, \qquad (3)$$

and find s_c by the formula,

$$s_c=\frac{6\,M}{b\,d^2\left[3\,k-k^2+\frac{6\,p'n}{k}\left(k-\frac{d'}{d}\right)\left(1-\frac{d'}{d}\right)\right]} \qquad (4)$$

Next find s_s by the formula,

$$s_s=\frac{M}{p\,j\,d^2\,b}=n\,s_c\frac{1-k}{k}, \qquad (5)$$

and find s_s' by the formula,

$$s_s' = n\,s_c \frac{k - \frac{d'}{d}}{k} \qquad (6)$$

36. The formulas giving the value of z and $j\,d$ need not always be solved, because formula **4**, Art **35**, does not include these terms and formula **5**, Art **35**, in one form, does not include them. Therefore, as a rule, the equations may be solved in the following order:

$$k = \sqrt{2\,n\left(p + p'\frac{d'}{d}\right) + n^2(p+p')^2} - n(p+p') \qquad (1)$$

$$s_c = \frac{6\,M}{b\,d^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \qquad (2)$$

$$s_s = n\,s_c \frac{1-k}{k} \qquad (3)$$

$$s_s' = n\,s_c \frac{k - \frac{d'}{d}}{k} \qquad (4)$$

37. As an example, assume that a beam is on a 20-foot span. It is limited in total depth to 20 inches and in width to 10 inches. It carries 1,000 pounds per foot besides its own weight. Design the beam.

First, the bending moment must be found. The beam weighs $\frac{10 \times 20 \times 150}{144} = 208$ pounds per foot. The bending moment is therefore $\frac{(1{,}000 + 208)\ 20 \times 20 \times 12}{8} = 724{,}800$ inch-pounds. The approximate value of A is found by the formula $s_s = \frac{M}{A\,j\,d}$. If d is taken as 18 inches, it becomes $16{,}000 = \frac{724{,}800}{A \times \frac{7}{8} \times 18}$. Therefore, $A = 2.876$ square inches, which is the amount of steel to be used at the bottom of the beam.

The critical value is $p=.00769$. This area is $p\,d\,b=.00769\times 18\times 10=1.384$ square inches. The excessive area is, then, $2.876-1.384=1.492$ square inches. The approximate amount of steel required at the top of the beam is then $1.492\times 2\frac{1}{4}=3.357$ square inches. The beam is now approximately designed, but it must be investigated to see whether or not it is safe. The values are $b=10$, $d=18$, $A=2.876$, $A'=3.357$, and $d'=2$. Then, $p=\frac{2.876}{10\times 18}=.01598$; also, $p'=\frac{3.357}{10\times 18}=.01865$.

Substituting the correct values in formula **1**, Art. **36**,

$$k=\sqrt{2\times 15\,(.01598+.01865\times\tfrac{2}{18})+15^2\,(.01598.+01865)^2}-15\,(.01598+.01865)=.3813$$

Substituting the values in formula **2**, Art. **36**, $s_c=$

$$\frac{6\times 724{,}800}{10\times 18^2\left[3\times .3813-.3813^2+\frac{6\times .01865\times 15}{.3813}(.3813-\tfrac{2}{18})(1-\tfrac{2}{18})\right]}=653$$

This value is 3 pounds higher than that recommended by the Joint Committee. However, the calculations do not have to be remade, as the value is close. When the beam is drawn on the plans, a little extra steel can be put in the top to reduce this compression if desired.

The value of s_s may be found by substituting the correct values in formula **3**, Art. **36**. Thus,

$$s_s=15\times 653\,\frac{1-.3813}{.3813}=15{,}893 \text{ pounds per square inch,}$$

which is slightly low, but practically correct.

The value of s_s' is found by substituting values in formula **4**, Art. **36**. Thus,

$$s_s'=15\times 653\,\frac{.3813-\frac{2}{18}}{.3813}=6{,}941 \text{ pounds per square inch}$$

This value is low, but it will always be found this way, and is the reason that double reinforced beams are not economical. This value cannot be raised in any convenient manner.

EXAMPLE FOR PRACTICE

A certain beam is built on a span of 30 feet. It is 12 inches wide and 20 inches deep. It contains 4.09 square inches of steel 2 inches from the bottom and 3.38 square inches of steel 2 inches from the top. It carries besides its own weight a load of 500 pounds per linear foot. There is doubt as to whether the stresses in the beam are too high. Determine the stresses in the concrete and in the steel in compression and tension.

$$\text{Ans.}\begin{cases} s_c = 775 \text{ lb. per sq. in.} \\ s_s = 15{,}770 \text{ lb. per sq. in.} \\ s_s' = 8{,}581 \text{ lb. per sq. in.} \end{cases}$$

DESIGN OF T BEAMS

38. Very often, in reinforced-concrete work, a beam and a floor slab are cast together. In such cases, the slab takes part of the compressive stresses caused by the bending moment, and the beam is no longer a rectangular beam, but one of **T** section.

Particular care must be exercised in the construction of **T** beams. The beam and the slab must be placed at one time so that there will not be a line of weakness between them. The stirrups must extend up into the slab, and the slab reinforcement should be tied to them. Short rods are sometimes run through the ends of the stirrups and made to extend into the concrete slab a short distance on each side of the beam. In short, every precaution possible must be used to insure that the slab and the beam will act together. If there is the slightest doubt about this unison, the beam must be figured as an ordinary rectangular beam underneath the slab.

39. The Joint Committee in discussing **T** beams states that "in beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

"Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

"(*a*) It shall not exceed one-fourth of the span length of the beam.

"(*b*) Its overhanging width on either side of the web shall not exceed four times the thickness of the slab.

"In the design of **T** beams acting as continuous beams, due consideration should be given to the compressive stresses at the support."

This last point is a matter of importance that is likely to be overlooked. At the supports of continuous beams there is a reverse bending moment, which is considered equal to the positive moment at the center of the span. Now, at the center of the span, the compression is at the top of the beam, and is taken up partly by the adjoining slab. At the support, however, inasmuch as the bending moment is reversed, the same amount of compression must be taken up by the narrow, or stem, part of the **T** at the lower side of the beam. As the total compression here is considered to be the same as at the middle of the span, the unit compression at the support is greater because the resisting area is less. The Joint Committee allows stresses 15 per cent. higher here than elsewhere; that is, if 650 pounds per square inch is allowed elsewhere, 748, or practically 750, pounds per square inch is allowed in compression of continuous beams near the supports. If this pressure is exceeded or if it is desired to reduce this value—some authorities consider it too high—then the beam must be made either wider or deeper in the stem of the **T** or else sufficient steel must be left in the bottom of the beam near the supports, making it practically a double reinforced beam at this point. This last method is the one usually followed.

40. In designing **T** beams, the same notation will be used as before, with the following changes:

b = width of flange, in inches;
b' = width of stem, in inches;
t = thickness of flange, in inches.

It is to be noted that the term *b* means the total width, and not the width of the beam proper, as in the preceding cases. This change is made so that some of the formulas already given will apply to **T** beams, as will be explained.

In the first place, the span of the beam and the thickness of the slab are known. Then, the depth of the beam and the width of the stem are usually assumed. From these conditions and the rules given, *b* may be assumed, but care should be taken not to consider the same part of the slab as acting for both of two adjacent beams. Then *b*, *t*, *d*, and *b′* are known. Reference to Fig. 4 will give a clear idea of where these dimensions are located.

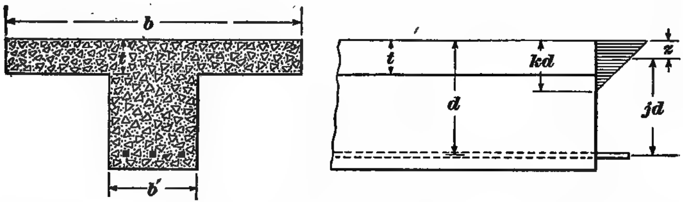


FIG. 4

41. First find an approximate value for *A* by the following formula:

$$A = \frac{M}{(d - \frac{1}{2}t)\, s_s} \qquad (1)$$

This formula will usually give values of *A* that err slightly on the side of safety. Some engineers, after *A* is thus found, consider the beam as designed. This method of procedure, however, is liable to cause trouble, because the value of *A* found by formula **1** may be such that the stress in the concrete is excessive. In such cases, the beam should be reassumed in larger proportions. Therefore, for a safe analysis of the beam, the following procedure must be carried out. This method neglects the compression in the stem.

After *A* has been assumed from formula **1** find the value of *k d* from the formula

$$k\,d = \frac{2\,n\,d\,A + b\,t^2}{2\,n\,A + 2\,b\,t} \qquad (2)$$

From here on, the work divides itself into two cases. Case I is when $k\,d$ is less than t; that is, when the neutral axis comes in the slab. Case II is when $k\,d$ is greater than t; that is, when the neutral axis comes in the stem of the **T**. If the problem is in Case I, the beam is investigated by the formulas used for a rectangular beam. It is to be remembered that formula **2** just given is meant for use in Case II. If the problem is in Case I, it simply indicates that fact, and the value of k must be assumed or found by the formulas for rectangular beams. If the problem falls in Case II, the following procedure is employed: Find z by the formula,

$$z=\frac{3\,k\,d-2\,t}{2\,k\,d-t}\times\frac{t}{3} \qquad (3)$$

Then find $j\,d$ by the formula,

$$j\,d=d-z; \qquad (4)$$

find s_s by the formula,

$$s_s=\frac{M}{A\,j\,d}, \qquad (5)$$

and find s_c by by the formula,

$$s_c=\frac{M\,k\,d}{b\,t\,(k\,d-\frac{1}{2}\,t)\,j\,d}=\frac{s_s}{n}\times\frac{k}{1-k} \qquad (6)$$

The values of s_s and s_c must be safe. As previously stated, these formulas neglect the compression in the stem. For approximate results, the formulas for rectangular beams are sometimes used.

42. The design of **T** beams is best illustrated by an example as follows: A beam is on a span of 16 feet. It carries a load of 5,000 pounds per foot, which includes its own weight. It is cast solid with a slab that is 10 inches thick. The total depth of the beam is 31 inches, and the width of the beam proper is 16 inches. Design the beam.

First, let it be decided to keep the steel up 2 inches from the under side of the beam. Then, $d=31-2=29$ inches. The maximum bending moment is $\frac{5{,}000\times16\times16\times12}{8}$

=1,920,000 inch-pounds. Substituting the correct values in formula **1**, Art. **41**,

$$A=\frac{1{,}920{,}000}{(29-\frac{10}{2})\ 16{,}000}=5 \text{ square inches}$$

The span is 16 feet. According to the recommendations of the Joint Committee, the value taken for b must not exceed $\frac{1}{4}\times 16=4$ feet, or 48 inches. Also, according to the Joint Committee, the overhang must not exceed $4\,t$, or $4\times 10=40$ inches. This second condition would make b equal to $40\times 2+16=96$ inches. As the former condition imposes the smaller value, it must be used. Some engineers also impose a third condition, namely, that b must be less than $5\,b'$; but as $5\,b'=5\times 16=80$, the conclusion arrived at above would not be altered. Therefore, b will be taken at 48 inches.

Substituting the correct values in formula **2**, Art. **41**,

$$k\,d=\frac{2\times 15\times 29\times 5+48\times 10^2}{2\times 15\times 5+2\times 48\times 10}=8.243 \text{ inches}$$

This value of $k\,d$ is less than the value of t. Therefore, the neutral axis lies in the flange and the problem falls in Case I, which means that it should be solved by the formulas for rectangular beams. In this solution, it is better to solve for k and j rather than assume one to be $\frac{3}{8}$ and the other to be $\frac{7}{8}$, because it is probable that the stresses in the concrete will be so low that these values will not be very close. Following this method,

$$p=\frac{A}{b\,d}=\frac{5}{48\times 29}=.003592$$

$$k=\sqrt{2\,p\,n+(p\,n)^2}-p\,n=\sqrt{2\times .003592\times 15+(.003592\times 15)^2}-.003592\times 15=.2788$$

$$j=1-\tfrac{1}{3}\,k=.9071$$

and
$$s_s=\frac{M}{A\,j\,d}=\frac{1{,}920{,}000}{5\times .9071\times 29}=14{,}597,$$

which is more than safe. As was stated before, the formula $A=\dfrac{M}{(d-\frac{1}{2}\,t)\,s_s}$ usually gives values that err on the side of safety, and this fact is evident from the result just obtained.

$$s_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\times 1{,}920{,}000}{.9071\times .2788\times 48\times 29^2}$$
$$= 376 \text{ pounds per square inch}$$

This value is low, but it is always best to have this value low in **T** beams.

43. As an example, to illustrate Case II, the following problem is proposed. It is very similar to the problem just given. A **T** beam is on a span of 16 feet. It carries a load of 5,000 pounds per foot, which includes its own weight. It is built solid with a slab 6 inches thick and the total depth of the beam is 31 inches. The value of d is taken at 29 inches, and b' is equal to 16 inches. Design the beam.

The bending moment is the same as before, namely, 1,920,000 inch-pounds. The value of A may be found approximately from the formula $A = \frac{M}{(d - \frac{1}{2}\,t)\,s_s}$. Thus, it is

$$A = \frac{1{,}920{,}000}{(29-3)\,16{,}000} = 4.6154 \text{ square inches}$$

The value of b is the same as before. Substituting the correct values in formula **2**, Art. **41**,

$$k\,d = \frac{2\times 15\times 29\times 4.6154 + 48\times 6^2}{2\times 15\times 4.6154 + 2\times 48\times 6} = 8.0388 \text{ inches,}$$

which is greater than the given value for t.

Substituting the proper values in formula **3**, Art. **41**,

$$z = \frac{3\times 8.0388 - 2\times 6}{2\times 8.0388 - 6}\times\frac{6}{3} = 2.4 \text{ inches}$$

Substituting the proper values in formula **4**, Art. **41**,

$$j\,d = 29 - 2.4 = 26.6 \text{ inches}$$

Substituting the proper values in formula **5**, Art. **41**,

$$s_s = \frac{1{,}920{,}000}{4.6154\times 26.6} = 15{,}639 \text{ pounds per square inch}$$

which is safe.

Substituting the proper values in formula **6**, Art. **41**,

$$s_c = \frac{1{,}920{,}000 \times 8.0388}{48 \times 6(8.0388 - \frac{6}{2})26.6} = 400 \text{ pounds per square inch,}$$

which is safe.

EXAMPLES FOR PRACTICE

1. A **T** beam is on a span of 24 feet. It carries a load of 8,000 pounds per foot, which includes its own weight. The following values have been decided on: $t = 10$ inches; $d = 41$ inches; $b' = 20$ inches; $n = 15$; and $A = 12$ square inches. Find the stress in the concrete and the steel.

$$\text{Ans.} \begin{cases} s_s = 15{,}502 \text{ lb. per sq. in.} \\ s_c = 438 \text{ lb. per sq. in.} \end{cases}$$

2. A **T** beam is on a span of 18 feet. It carries a load of 4,000 pounds per foot, which includes its own weight. The following values have been decided on: $t = 6$ inches; $d = 30$ inches; $b' = 16$ inches; $n = 15$; and $A = 4\frac{1}{2}$ square inches. Find the stress in the concrete and the steel.

$$\text{Ans.} \begin{cases} s_s = 15{,}630 \text{ lb. per sq. in.} \\ s_c = 357 \text{ lb. per sq. in.} \end{cases}$$

44. As was stated, the formulas for Case II already given, neglect the compression in the stem of the **T**. They will usually be found to give satisfactory values. However, when the slab, or flange part, of the beam is small, evidently a good part of the compressive resistance is taken up by the stem. To neglect this compression would make the concrete appear more heavily stressed than it really is; also, when a large part of the compression is taken by the stem, the value of $j\,d$ is really less than that calculated by the formulas already given and the stress in the steel will be somewhat higher than was supposed. The Joint Committee recommend that the following formulas be used if the flange is small compared with the stem:

$$k\,d = \sqrt{\frac{2\,n\,d\,A + (b - b')\,t^2}{b'} + \left(\frac{n\,A + (b - b')\,t}{b'}\right)^2} - \frac{n\,A + (b - b')\,t}{b'} \qquad (1)$$

$$z = \frac{(k\,d\,t^2 - \frac{2}{3}\,t^3)\,b + (k\,d - t)^2\left[t + \frac{1}{3}(k\,d - t)\right]b'}{t\,(2\,k\,d - t)\,b + (k\,d - t)^2\,b'} \qquad (2)$$

$$j\,d = d - z \tag{3}$$

$$s_s = \frac{M}{A\,j\,d} \tag{4}$$

$$s_c = \frac{2\,M\,k\,d}{\left[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b'\right] j\,d} \tag{5}$$

These formulas are rarely used; nevertheless, when employing the formulas under Case II, it must be borne in mind that the compression in the stem is neglected.

45. As an example of the use of the formulas in the preceding article, the problem given in Art. **43** will be solved by them. The following values are the same in both cases:

$M = 1{,}920{,}000$ inch-pounds;
$t = 6$ inches;
$d = 29$ inches;
$b' = 16$ inches;
$A = 4.6154$ square inches;
$b = 48$ inches.

Therefore, $k\,d$

$$= \sqrt{\frac{30 \times 29 \times 4.6154 + (48-16)6^2}{16} + \left(\frac{15 \times 4.6154 + (48-16)6}{16}\right)^2} - \frac{15 \times 4.6154 + (48-16)6}{16}$$

$$= \sqrt{322.962375 + 266.568888} - 16.3269 = 7.955 \text{ inches}$$

Then,

$$z = \frac{(7.955 \times 6^2 - \tfrac{2}{3}\,6^3)48 + (7.955-6)^2\left[6 + \tfrac{1}{3}(7.955-6)\right]16}{6(2 \times 7.955 - 6)48 + (7.955-6)^2\,16}$$

$$= 2.484 \text{ inches}$$

$$j\,d = 29 - 2.484 = 26.516 \text{ inches}$$

and $s_s = \dfrac{1{,}920{,}000}{4.6154 \times 26.516} = 15{,}689$ pounds per square inch

This is slightly larger than the value found in Art. **43**, but is not dangerously so.

$$s_c = \frac{2 \times 1{,}920{,}000 \times 7.955}{\left[(2 \times 7.955 - 6)48 \times 6 + (7.955-6)^2\,16\right]26.516}$$

$= 395.2$ pounds per square inch,

which is a little less than the value in Art. **43.** On the whole, for cases like those given, it can be seen that the formulas that neglect the compression in the stem are sufficiently accurate unless the beam proper is large in proportion to the slab.

DETAILS OF DESIGN

46. Fireproofing.—So far, the quantity of concrete below the reinforcement in the lower part of the beam has not been discussed. This concrete serves two purposes. It holds the reinforcement in place and protects it from fire and moisture. The Joint Committee recommends that to protect the steel from fire the thickness of concrete under the reinforcement of girders should be 2 inches; under the reinforcement of beams, 1½ inches; and under the reinforcement of floor slabs, 1 inch. These recommendations are for ordinary conditions, and must be changed as circumstances require. All sharp corners of beams should be chamfered.

47. Splices.—In splicing bars that take tension, the splice must be such that the tension of the entire bar is taken by the bond between that bar and the concrete, unless, of course, the bars are securely fastened together.

A very simple rule can be derived for the amount of lap necessary. Its derivation will be readily understood by referring to Fig. 5. Here, two round bars are lapped, and

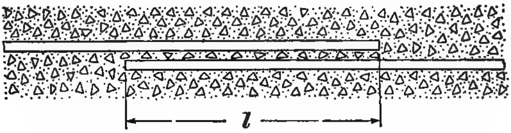


FIG. 5

the value of l must be found to make the section *good*, as it is called. Let the diameter of the bar, in inches, be called d. The circumference of the bar is then $d \times 3.1416$, and its surface at the lap is $d \times l \times 3.1416$. If the safe adhesion is 80 pounds per square inch, the total adhesion will be $d \times l \times 3.1416 \times 80$. The distance l should be such that the bar is liable to break before it will slip; that is, the resistance to slipping must at least equal the resistance to rupture. The area of the bar is $d^2 \times .7854$ square inch. If the unit tensile stress is taken at 16,000 pounds per square inch, the resistance to rupturing will be $d^2 \times .7854 \times 16{,}000$. Equating these two values, it is found that $d \times l + 3.1416 \times 80 = d^2 \times .7854 \times 16{,}000$, which reduces to $l = 50\,d$. Therefore, in lapping rods in reinforced-concrete work, the lap should be made fifty times the diameter of the rod. If the rods are made of drawn wire, the lap should be twice this amount. These values of safe bond are the ones recommended by the Joint Committee for concrete; that will develop a strength of 2,000 pounds in 28 days, tested as previously stated. Of course, for deformed bars, the lap does not have to be so great.

48. Value of n.—The value of n in the preceding formulas was taken as 15. This value of n is known to increase somewhat with the stress in the concrete, but many designers consider it to be constant. If the value of E_s is 30,000,000, then the value of E_c, to make n equal to 15, will be 2,000,000. The value to be used for n varies with different building laws and in the opinion of different engineers. The value employed in the examples is the one frequently used, and the one that the Joint Committee recommends to be used under ordinary conditions for the grade of concrete just mentioned.

49. Size and Spacing of Bars.—In the preceding formulas, the value of A has been taken as exact. In actual practice, this area of rods must be made up of commercial stock that is rolled in standard sizes. It will seldom be found possible to make up the exact area required. If, for this reason, slightly less steel than is specified is used, the calculations should be remade in order to make sure that the stresses in the steel will not be excessive.

In placing the bars in the concrete, it should be remembered that they must be protected from fire at the sides of the beam as well as underneath. In other words, if $1\frac{1}{2}$ or 2 inches of concrete is allowed below the steel, these rods should be so placed that the same quantity of concrete will be between them and the sides of the beam; also, they should be placed far enough apart to allow the concrete to flow easily around them.

The general recommendations of the Joint Committee are in part as follows: "The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than ½ inch."

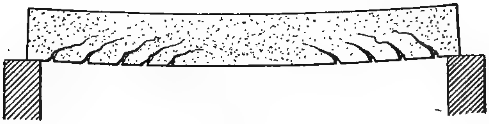


FIG. 6

50. Shear in Beams.—One of the most frequent methods of failure in concrete beams is illustrated in Fig. 6. This form of breaking is called *failure by diagonal shear*, but it is generally due to *failure by diagonal tension*. The theory of this form of failure will not be discussed here, but it may be said to be due to the facts that all the reinforcement is in the bottom of the beam and that no truss rods or stirrups are used. The use of *truss rods* and *stirrups* will help prevent this form of failure, and both are used in beams of considerable depth.

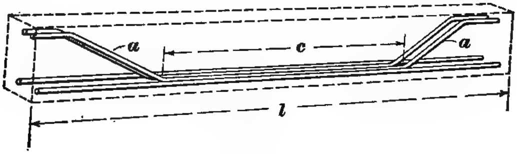


FIG. 7

51. Truss rods, besides preventing the diagonal cracking just mentioned, serve to take up at least part of the negative bending moment that occurs in continuous beams over the supports; also, if bent up high enough, they help tie the beam and slab together.

Fig. 7 shows the main rod reinforcement for an ordinary beam that is either carrying a uniformly distributed load or a load concentrated at its center. The maximum bending moment occurs at the center of the span; therefore, all the reinforcement is found at the bottom of the beam. However, near the ends of the span, the bending moment becomes less and the tendency to diagonal tension becomes greater. Therefore, some of the reinforcement can be spared from the bottom of the beam and may be bent up to form truss rods, as shown at *a*. These truss rods run directly across the place where the tension cracks would appear; and, thus, for the beam to fail here, the truss rods would have to be ruptured.

The distance designated by *c* in the illustration, is often made such that some of the steel will pass through the neutral surface at the point of contraflexure; that is, when the bending moment changes from positive to negative. It usually varies so that *c* is from one-fourth to one-half of *l*. It is understood that the truss rods, although shown for convenience as stopping short at the end of the span, extend into the adjoining beam or are bent over or secured at the ends so as not to slip. They also are preferably in smaller unit bent up, not all at once, but in pairs at intervals from the bottom of the beam to distribute their effect.

52. Stirrups, besides preventing diagonal tensile cracks, serve two other purposes, namely, to tie the reinforcement into the body of the beam in case the concrete below should fail from fire or any other cause, and to tie the beam to the slab, for they not only enter the slab, but may also be made to tie up to its reinforcement.

There is no rigid analysis governing the size and placing of stirrups. The matter is governed largely by experience. Stirrups are often made of ¾-inch square or $\frac{3}{16}'' \times 1''$ iron. They are spaced closer at the ends of the beam than at the center of the span. The Joint Committee recommends a maximum spacing of three-quarters of the depth of the beam. This maximum spacing occurs at the center of the span. At the ends of the span, the stirrups are often spaced one-sixth or one-seventh of the depth of the beam. These stirrups must be spaced with extreme care, as the strength of the structure depends on them. Designs of successful similar buildings will often serve as models, not to be directly copied, perhaps, but to indicate at least about what the size and the spacing of stirrups should be. Short, deep beams and **T** beams must be designed with especial care.

If the stirrups are inclined to the vertical, they should be securely fastened to the main reinforcement. Care should be taken to see that the stirrups will not pull out of the concrete. Often, they are bent over at the ends or securely fixed to the slab reinforcement. Deformed bars also give a better grip on the concrete.

53. In regard to web resistance, the Joint Committee states that calculations should be made on a basis of maximum shearing stress; that is, although the beam fails by diagonal tension, it is designed to withstand shear. Therefore, the formulas, although using shear, must really limit the diagonal tension. This would indicate that exact methods for calculating these stresses are not yet devised. The Joint Committee states that, "Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made."

The formulas referred to by the Joint Committee employ the following notation:

V = total shear, in pounds, to be found as explained in *Forces Acting on Beams;*
v = shearing unit stress, either horizontal or vertical, in pounds per square inch;
b = breadth of beam, in inches;
$j\,d$ = lever arm of resisting couple, as explained before, in inches;
P = stress in one stirrup, in pounds per square inch;
a = horizontal spacing of stirrups, in inches;
b' = width of stem of **T** beam, in inches.

The formulas for rectangular beams, whether reinforced at the bottom only or double reinforced, are:

$$v=\frac{V}{b\,j\,d} \qquad (1)$$

For vertical stirrups:

$$P=\frac{V\,a}{j\,d} \qquad (2)$$

For stirrups inclined at 45°:

$$P=.7\,\frac{V\,a}{j\,d} \qquad (3)$$

For **T** beams:

$$v=\frac{V}{b'\,j\,d} \qquad (4)$$

54. As an example, assume that the main reinforcement in a certain beam has been designed and that it remains to look into the web reinforcement. The beam is on a 20-foot span and carries 1,500 pounds per foot, which includes its own weight. Assume that $b=12$ inches and that $d=21$ inches.

First, the maximum shear is equal to either reaction, or

$$\frac{1{,}500\times 20}{2}=15{,}000 \text{ pounds}$$

The unit shear, assuming that $j=\frac{7}{8}$, is

$$v=\frac{V}{b\,j\,d}=\frac{15{,}000}{12\times\frac{7}{8}\times 21}=68 \text{ pounds per square inch}$$

The Joint Committee states that for concrete alone the web shear should not exceed 40 pounds per square inch, with truss rods properly placed, 60 pounds per square inch and that, even with truss rods and stirrups, the shear should never be over 120 pounds. As the value found is less than 120 pounds, the beam may be used; but as it is more than 40 pounds, stirrups and truss rods must be employed.

The size of the stirrups is next selected, and the allowable stress in it is decided on. Thus, the value of P, the allowable stress in one total stirrup, may be determined, bearing in mind that either the stirrup must have some mechanical bond or else the value of P must not be great enough to disengage the stirrup from the concrete by either tension or shear. Let it be assumed in this case that the allowable stress in each stirrup is 6,000 pounds. Now, it is generally assumed that the concrete itself can take one-third of the shear; therefore, the stirrups have to resist only two-thirds of the shear; that is, $\frac{2}{3}\times 15{,}000 = 10{,}000$ pounds. Substituting the current values in formula **2**, Art. **53**, it becomes $6{,}000 = \frac{10{,}000\, a}{\frac{7}{8}\times 21}$, or $a = 11$ inches, about. This is the spacing for vertical stirrups near the supports where the shear is greatest. Of course, near the center of the beam, this spacing will increase as the shear decreases, but it should never be greater than $\frac{3}{4}\, d$.

It must be remembered that this method of designing web reinforcement is not a complete analysis of the case, and the Joint Committee does not state that it is more than tentative. However, it serves as a guide, and its use should be supplemented by good sound judgment and experience.

One point recommended by the Joint Committee to assist in preventing diagonal tension cracks is to arrange the horizontal reinforcement so that its unit stresses will be relatively low at points of high shear.

55. Another detail of design that is generally considered at the same time that the question of stirrups is taken up, is the bond stress of the concrete to the reinforcement. It is not alone necessary that the bar be long enough to offer bond to withstand the total tension in the reinforcement; in addition, the unit bond at any one point must not be exceeded. It is evident that if the tension in the main reinforcement at the center of the span is, say T, and the tension in it at another section near it is T', the difference in tension between these two points will be $T - T'$. This extra tension, which the bar looses between these two points, is of course taken up by the bond between the bars and the concrete, and this bond must not be exceeded. Rather than find the tension at two points by finding the bending moment, a simpler method has been devised.

Let u = unit bond at section under consideration, in pounds per square inch;
O = sum of perimeters of horizontal reinforcement bars at the section under consideration;
$d\,j$ = the lever arm, as before;
V = external shear at the section under consideration.

Then,
$$u = \frac{V}{j\,d\,O}$$

The value of u thus found must not exceed safe limits, which, according to the Joint Committee, are 80 pounds for plain bars and 40 pounds for drawn wire for the grade of concrete recommended. Of course, for deformed bars, higher bond values may be used.

COLUMNS

CONCENTRICALLY LOADED COLUMNS

56. There are two general methods of reinforcing concrete columns with steel. One method is known as *straight reinforcement* and the other as *hooped reinforcement*. These two styles of reinforcement are illustrated in Fig. 8. In (*a*) is shown straight reinforcement. It consists of steel rods that stand vertically in the concrete. Sometimes, the rods are placed directly in the middle of the column, but as a rule they are arranged around the outside of the column at least 2 inches from the surface. These steel rods are tied together with wire ties, as shown at *a*. The distance between two ties should not exceed the width, or diameter, of the column. If the ties are spaced too far apart, the column is liable to fail by the reinforcement bulging out, as shown in Fig. 9.

In Fig. 8 (*b*) is shown a column reinforced with hooped reinforcement. This type of reinforcement consists of either a steel spiral or a separate steel hoop that is at least 2 inches from the surface of the column, as shown at *b*.

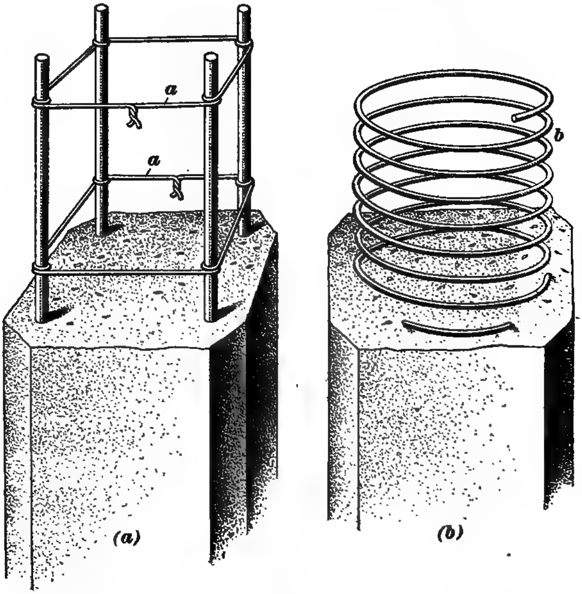


FIG. 8

57. In addition to the two types of reinforcement just mentioned there are many styles, more or less indeterminate, that engineers must examine with care. Thus, Fig. 10 (*a*) shows a column in which the reinforcement is straight. The netting around the reinforcement serves the twofold purpose of holding the reinforcement in place and of holding the surface of the concrete so that it will neither crack nor crumble off during a fire.

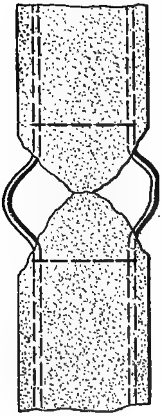
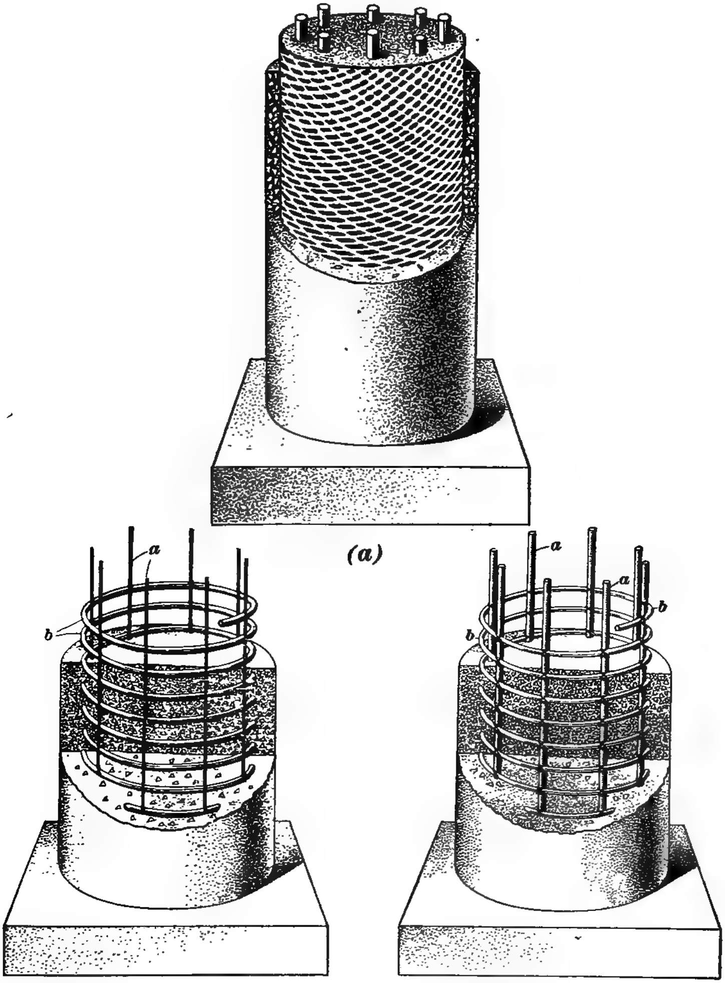


FIG. 9

In view (*b*) is shown a hooped column. The hooping is shown at *b*, and at *a* are shown light steel wires that hold the hooping in place while the concrete is being poured. These wires *a* are too thin to be considered as vertical reinforcement.

In (*c*) is shown a column that has two distinct kinds of reinforcement, the hooped reinforcement at *b* and the vertical, or straight, reinforcement at *a*.



(*a*)

(*b*) (*c*)

Fig. 10

58. Column Fireproofing.—The Joint Committee recommend that in concrete columns the outside 1½ inches of concrete be not considered in the design, as it may be injured in a fire. This coating, therefore, is simply a *fireproof covering*. After a column is designed, provided the recommendations of the Joint Committee are followed, 1½ inches is added to all sides; that is, if the column is rectangular, it is made 3 inches broader and 3 inches deeper. All sharp corners should be chamfered off. Some engineers do not add this fireproof covering, although it is safe to do so.

59. Straight Reinforcement.—The design of **straight reinforcement** in columns will be taken up first.

Let A_c = effective area of cross-section of concrete, in square inches;
A_s = area of cross-section of steel, in square inches;
E_c = modulus of elasticity of concrete;
E_s = modulus of elasticity of steel;
s_c = compressive stress per unit area of concrete;
s_s = compressive stress per unit area of steel;
m_c = compressive unit strain in concrete;
m_s = compressive unit strain in steel;
$n = \frac{E_s}{E_c}$;
W = total load on column.

The load on the column equals, of course, the total compressive stress on the concrete plus the total compressive stress on the steel. Expressed as a formula, this would be

$$W = A_c s_c + A_s s_s \qquad (1)$$

From the principles given in *Stresses and Strains*,

$$E_c = \frac{s_c}{m_c} \text{ and } E_s = \frac{s_s}{m_s}$$

These equations may be written

$$m_c = \frac{s_c}{E_c} \text{ and } m_s = \frac{s_s}{E_s}$$

Now, as the concrete and steel are in direct contact and there is no slip of the steel—that is, the bond between the steel and concrete is good—it follows that if the concrete in the column is compressed, say $\frac{1}{50}$ inch, the steel will also be compressed a like amount. As the same unit compression is exerted on both the steel and the concrete throughout the entire column, and the steel is bonded all the way, the unit strain in the concrete must equal the unit strain in the steel; that is, m_c must be equal to m_s. Therefore,

$$\frac{s_s}{E_s}=\frac{s_c}{E_c},$$

or

$$s_s=s_c\frac{E_s}{E_c}=s_c\,n \qquad (2)$$

Substituting the value just found for s_s in formula **1**,

$$W=A_c\,s_c+A_s\,s_c\,n,$$

or

$$W=s_c\,(A_c+n\,A_s) \qquad (3)$$

This is the equation used to design columns with straight reinforcement.

The Joint Committee recommends a value for s_c of 450 for concrete that develops a unit compressive strength of 2,000 pounds in 28 days, tested as previously stated. Some engineers do not use a value so high, but it will be used throughout this Section in order to get uniform results. The value of n is taken at 15, although sometimes considered less.

60. As an example, assume a column 1 foot square with 4 square inches of steel. What safe load will it carry?

First, a 1½-inch covering must be deducted. This makes the column $9\times 9=81$ square inches. Deducting the area of the steel, $A_c=81-4=77$ square inches. Also, $A_s=4$ square inches. Substituting these values in formula **3**, Art. **59**,

$$W=450\times(77+15\times 4)=61{,}650 \text{ pounds}$$

61. It will be well to investigate the theory of straight steel reinforcement in columns in order to obtain some idea of the economy of the system used. As was stated, the safe value for s_c is taken at 450 pounds per square inch; also, n is considered to be equal to 15. Substituting these values in formula **2**, Art. **59**,

$$s_s = s_c\, n = 450 \times 15 = 6{,}750 \text{ pounds per square inch}$$

Now, the safe unit compressive stress for steel may be taken as high as 16,000 pounds. Therefore, in a column with straight reinforcement, and the grade of concrete mentioned, the steel cannot be stressed up to its safe limit without overstressing the concrete. It cannot be said, therefore, that this method of placing the steel is economical. The use of steel in the column has advantages. In the first place, the formulas given in Art. **59** do not take into account the length of the column; the longer a column is, the more it is likely to bend or bulge, and the steel helps materially to resist this tendency. Then, again, the introduction of steel permits the column to be made much smaller in size, and this is often of great advantage. The most important advantage of the steel, however, is that there are usually certain slight eccentricities of load that are impossible to predetermine, and the steel more or less takes care of them.

The length of a column of this kind, according to the recommendations of the Joint Committee, should be less than 15 times its diameter or least width.

62. Having shown how to investigate a column, the method of designing one to carry a certain load will now be considered. Thus, assume that it is desired to design a column to carry a load of 40 tons.

In the first place, three things must be decided: (1) The value to be used for s_c; (2) that to be used for n; and (3) that to be used for the percentage of reinforcement. The values to be used for s_c and n will be chosen at 450 and 15, respectively. The percentage of reinforcement may be any reasonable amount. Let it be supposed that it is desired not to have the column too large; therefore, assume 2 per cent. In the examples for practice that follow, to obtain uniform results, the three values just given will also be used. To return to the problem, let A_c = area of concrete. Then, A_s = 2 per cent. of $A_c = \frac{1}{50} A_c$. Substituting the correct values in formula **3,** Art. **59,** it will be found that

$$W = s_c\,(A_c + n\,A_s)$$
$$= 450\,(A_c + 15 \times \tfrac{1}{50}\,A_c)$$
$$= 450\,A_c\,(1 + \tfrac{3}{10}) = 450 \times \tfrac{13}{10} \times A_c = 585\,A_c$$

But $W = 40 \times 2{,}000 = 80{,}000$ pounds; therefore,

$80{,}000 = 585\,A_c$ and $A_c = 80{,}000 \div 585 = 137$ square inches

The amount of steel required is $137 \times \frac{2}{100} = 2.74$ square inches. Therefore, the total area of the column will be $137 + 2.74 = 139.74$ square inches. A 12″ × 12″ column would therefore be ample for the conditions imposed. To this column, however, 1½ inches must be added on each surface for fireproofing purposes. Therefore, the column will be 15 inches square.

EXAMPLES FOR PRACTICE

1. Design a square column to carry a safe load of 48 tons.

Ans. { Size of col., approx. 16 in. on a side
Area of steel required, 3.3 sq. in.

2. A column is 14 inches in diameter and contains 4 square inches of steel. What safe load will it carry? Ans. 67,965 lb.

3. Design a round column to carry 62 tons safely.

Ans. { Diam., about 19.5 in.
Area of steel, 4.24 sq. in.

4. Design a rectangular column 10 inches wide to carry 32 tons safely.

Ans. { Depth, about 19 in.
Steel required, 2.19 sq. in.

63. Empirical Rules for Straight Reinforcement. There are one or two more or less empirical formulas that are used in designing concrete columns with straight reinforcement. These formulas mostly originate from the building laws of various cities. One rule stipulates that the safe allowable load shall be 500 pounds per square inch of column section properly reinforced, counting both concrete and steel the same. Suppose it is desired to design a column to carry 25 tons. Now, 25 tons equals 50,000 pounds, and the required area would be $50{,}000 \div 500 = 100$ square inches, which would mean a 10″ × 10″ column. The 1½-inch fire protection is not included in the preceding calculations. Either sufficient steel must be inserted in the column to prevent the concrete from being overstressed in case 1½ inches is removed from the outside by fire, or the 1½ inches must be added afterwards, or, as is sometimes done, it must be neglected altogether.

64. In a large building, the loads on the columns in the lower floors become very great, and as it is usually not desirable to increase the outside size of the column abnormally, the percentage of reinforcement is increased. A style of reinforcement sometimes employed under such conditions is shown in Fig. 11. It consists of four angles riveted back to back in the center of the column to form a steel core. The area of this steel core can be increased if desired by using packing plates between the angles. Some engineers assume that the steel core takes all the vertical load and that the concrete prevents it from bending sidewise. The rule for designing such a column, therefore, is to allow a safe compressive unit stress on the steel and nothing on the concrete. Thus, if 20 square inches is required and four angles are to be used, each one will have to have an area of 20 ÷ 4 = 5 square inches. Angles 4 in. × 4 in. × $\frac{11}{16}$ in. would be large enough. The stress in the steel should not be so high as to cause the concrete around it to break loose. In construction of this kind, steel rods should be used in conjunction with the steel core. These rods should be spaced as shown in the figure and securely wired together. They are not considered in calculating the safe load, but are inserted to stiffen the concrete, which is under high stress, and to prevent it from splitting away from the steel core in the center.

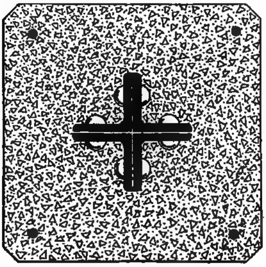


FIG. 11

65. Hooped Columns.—In regard to the effect of **hooped columns**, the Joint Committee states that: "The general effect of bands or hoops is to increase greatly the 'toughness' of the column and its ultimate strength, but hooping has little effect upon its behavior within the limit of elasticity. It thus renders the concrete a safe and more reliable material, and should permit the use of a somewhat higher working stress." That is to say, in hooped concrete columns the hooping itself is not used in calculating the strength, but on account of the hooping a higher unit stress in the concrete may be used.

The Joint Committee recommends a stress 20 per cent. higher than that used for longitudinal reinforcement. Thus, if 450 pounds is used for longitudinal reinforcement, the stress for hooped columns will be 540 pounds. The Committee specifies, however, that to use this increase of stress the volume of the steel hooping must be at least 1 per cent. of the volume of the enclosed concrete. Some engineers use stresses lower than this, but these mentioned are used in this Section to make results uniform. One point of importance, however, to be remembered is that the concrete outside of the hooping is assumed to take no stress, all the stress being taken by the enclosed concrete. As the steel is embedded in the concrete at least 2 inches, this amount, instead of 1½ inches, must be deducted to get the effective area of the column.

66. As an example, design a round-hooped concrete column to carry 100 tons. The load to be carried is $100 \times 2{,}000 = 200{,}000$ pounds. The required effective area is then $200{,}000 \div 540 = 370.37$ square inches. The diameter of the hooping, therefore, is approximately 22 inches. As there must be 2 inches of concrete outside of the hooping, the outside diameter must be at least 26 inches. The quantity of steel required per inch should be at least $370.37 \div 100 = 3.7$ cubic inches. In all cases, the hooping should be arranged in hoops, or spirals, so that the vertical distance from hoop to hoop, or from one loop of the spiral to another, will not be greater than one-fourth the diameter of the enclosed column. In this case, this maximum distance would be $22 \times \frac{1}{4} = 5\frac{1}{2}$ inches. If hoops are used, the volume of the steel in each hoop is $5\frac{1}{2} \times 3.7 = 20.35$ cubic inches. As the circumference of the hoop is 69 inches, the sectional area of each hoop must be $20.35 \div 69 = .2949$ square inches. The nearest size of round rod that can be easily obtained on the market is the $\frac{5}{8}$ rod. The values given for the spacing and the size of the hoops are the limiting values. Safer values may be used if considered necessary.

67. Combined Reinforcement.—Sometimes, both hooped and longitudinal reinforcement is used. The hooping must not be considered as before, but if more than 1 per cent. and less than 4 per cent. of longitudinal bars is used, the Joint Committee will allow stresses in the concrete 45 per cent. greater than when no hooping is used; that is, a unit stress of about 650 pounds.

68. As an example, design a round column to carry 75 tons. Use 3 per cent. of longitudinal bars. The total load on the column is $75 \times 2{,}000 = 150{,}000$ pounds. Substituting correct values in formula **3**, Art. **59**, it becomes $150{,}000 = 650\ (A_c + 15 \times A_s) = 650\ A_c\ (1 + .45)$. Therefore, $A_c = 159.2$ and $A_s = 4.776$. The total area inside the hoop reinforcement, then, is about 164 square inches. This corresponds to a diameter that is a trifle less than 14.5 inches. By adding 2 inches outside the reinforcement, the diameter of the column is 18.5 inches. The rings must have a volume of at least 1.64 cubic inches for every inch in the height of the column.

The values used throughout these column problems are recommended by the Joint Committee. They are based on concrete that will gain an ultimate strength of 2,000 pounds per square inch in 28 days, when tested as specified before. They are somewhat high and may be reduced to suit the building laws of various cities or the judgment of the engineer.

EXAMPLES FOR PRACTICE

1. Design a hooped column that has no longitudinal reinforcement, to carry $62\frac{1}{2}$ tons.

Ans. $\begin{cases}\text{Diam., } 21\frac{1}{4} \text{ in.}\\ \text{Metal per in., } 2.31 \text{ cu. in.}\end{cases}$

2. Design a hooped column having in addition to the hoops $2\frac{1}{2}$ per cent. of vertical reinforcement, to carry 80 tons.

Ans. $\begin{cases}\text{Diam., } 19\frac{3}{8} \text{ in.}\\ \text{Area of longitudinal reinforcement, } 4.48 \text{ sq. in.}\\ \text{Hooping per in. of height, } 1.79 \text{ cu. in.}\end{cases}$

ECCENTRICALLY LOADED COLUMNS

69. The methods of design just considered are for columns centrally loaded. Sometimes the column is not centrally loaded, in which event extra precautions must be taken. In ordinary building construction, where beams frame in opposite sides of a column, the column will be centrally loaded if both beams are loaded evenly. However, if one beam carries a large live load and the other beam does not, the loaded beam will deflect. This deflection will cause the column to bend. If the beams on each side and the columns have about the same rigidity, the bending moment in the column both above and below the beam will be about $\frac{W\,l}{48}$, in which W is the live load and l is the span. In the lower stories of a building, there will probably be such heavy loads from the stories above that the bending caused by the floorbeam is insignificant when compared with the concentric load from the superimposed columns. In the upper stories, however, this effect will be more noticeable, and should therefore always be borne in mind. The wall columns of a building offer still greater difficulties in calculation. Sufficient steel should be used and low unit stresses employed. If doubt exists about the safety, it should be investigated for bending.

70. The wind also causes stresses in columns. If the building is comparatively low and wide; or is protected, these stresses are either small or absent; but if the building is unprotected from the wind and is narrow, the wind stresses must be considered. Brackets under beams and girders are largely used to stiffen the joints of a structure against wind.

There is probably no exact way to calculate the stresses caused by the wind. The following formulas, however, which are based partly on an analogy of conditions to those in a steel building, will serve at least to indicate that wind stresses are present and of importance:

$$C = \frac{P x}{2 w} \qquad (1)$$

$$M = \frac{P y}{4} \qquad (2)$$

A diagram of the building is shown in Fig. 12. At *a* is shown the column under investigation. *P* is the total wind pressure over a surface whose height is *x* and whose width is the distance between two bays; *y* is the clear height of one story; and *w* is the distance from center to center of columns. *C* is the total concentric tension or compression put in the column by the wind, and *M* is the bending moment in the column. The wind also causes in the cross-girders a bending moment of about the same amount.

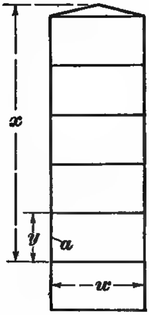
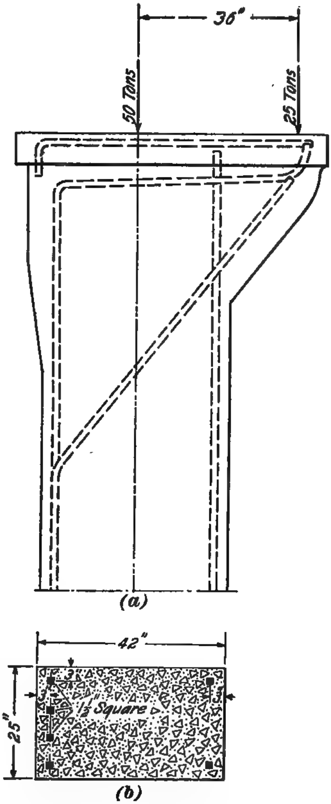


FIG. 12

71. An approximate method of design that will probably give safe results for a reinforced column that is eccentrically loaded is as follows: First, assume a column section. Then find the stress in the concrete and the steel due to compression, and next the stress in the concrete and the steel due to bending. Add the two stresses in the concrete thus found, and also add algebraically the two stresses in the steel thus found. The total stresses in the concrete and the steel thus obtained must be below the safe allowable stress for these materials. As this method is approximate, it will be well always to use low unit stresses. It is evident that hooped columns cannot resist tension on the side of the column opposite the eccentric load.

72. An example will serve to bring out these points. The column shown in Fig. 13 (*a*) carries two loads, one concentric and one eccentric, as shown. The column is of the size and design shown in the section (*b*). What stresses are produced?

First, the direct pressure will be found. It must be remembered that the outside 1½ inches of concrete is for fireproofing purposes only. Therefore, the total effective area of the column is 22 × 39 = 858 square inches. For the reinforcement shown, $A_s = 6 \times 2.25 = 13.50$ square inches. Therefore, $A_c = 858 - 13.50 = 844.5$ square inches. The total load W is $(50 + 25) \times 2{,}000 = 150{,}000$ pounds. Substituting correct values in formula **3,** Art. **59,** it becomes $150{,}000 = s_c\,(844.5 + 15 \times 13.50)$. Therefore, $s_c = 143$ pounds per square inch, compression, and $s_s = n\,s_c = 15 \times 143 = 2{,}145$ pounds per square inch, compression.



(*a*)

(*b*)

FIG. 13

The moment caused by the eccentric load may be taken at $36 \times 25 \times 2{,}000 = 1{,}800{,}000$ inch-pounds. Using the notation employed in connection with beams, and remembering to deduct for the fireproofing, $p = \frac{4 \times 2.25}{37.5 \times 22} = .01091$; $p' = \frac{2 \times 2.25}{37.5 \times 22} = .00545$; $d = 37.5$ inches; and $d' = 1.5$ inches. Therefore, according to formula **1,** Art. **36,**

$$k = \sqrt{2 \times 15\left(.01091 + .00545 \times \frac{1.5}{37.5}\right) + 15^2(.01091 + .00545)^2} - 15(.01091 + .00545) = .3824$$

From formula **2**, Art. **36**, $s_c=$

$$\frac{6\times 1{,}800{,}000}{22\times 37.5^2\left[1.1472-.3824^2+\frac{90\times .00545}{.3824}\left(.3824-\frac{1.5}{37.5}\right)\left(1-\frac{1.5}{37.5}\right)\right]}$$

$=245$ pounds per square inch

From formula **3**, Art. **36**,

$$s_s=15\times 245\times\frac{1-.3824}{.3824}=5{,}935 \text{ pounds per square inch}$$

From formula **4**, Art. **36**,

$$s_s'=15\times 245\times\frac{.3824-\frac{1.5}{37.5}}{.3824}=3{,}291 \text{ pounds per square inch}$$

Thus, the greatest compression in the concrete is $143+245=388$ pounds per square inch, and the greatest tension in the steel is $5{,}935-2{,}145=3{,}790$ pounds per square inch. The greatest compression in the steel is $2{,}145+3{,}291=5{,}436$ pounds per square inch. These stresses are low and therefore the column is apparently safe. If the stresses are considered too high, a larger section of column must be assumed, and the stresses must be refound. If the stresses are considered too low, a smaller section may be investigated.

73. There are some inaccuracies in the foregoing investigation that must not be overlooked. In the first place, owing to the fact that there is more steel on one side of the column than on the other, the central load is not absolutely central with respect to the resisting column surface, because the column is more rigid on one side than on the other. Often, the column is designed with the same quantity of steel on both sides. This tends to reduce this defect. Then, the moment of the eccentric load was taken about the center of the column. Bending really occurs about the neutral axis. This neutral axis, however, is not in the same place as was found for beams, because the neutral axis is the line along which there is no stress and the direct compression in the column would shift this line. A third inaccuracy is caused by the column bending out of plumb. This would have the effect of increasing the lever arm of the eccentric load. However, the column should be so short in comparison to its length that this deflection is negligible. Still another defect is that the space between the rods on the compression side is so wide that it is doubtful whether the column would act exactly as a double reinforced beam under bending. Lastly, the exact distribution of stress in the column is unknown. For all these reasons, eccentrically loaded columns cannot be investigated exactly and should not be used if they can be avoided. In any case, low unit stresses should be employed. The design of columns so loaded depends a great deal on experience, and they are not recommended to be used in important work.

In eccentrically loaded columns, particular attention must be paid to the wiring together of the rods. Here, the wires are used to prevent the bars from bulging, and they also act as stirrups to take up shear and diagonal tension. A reasonable amount of hooping will often answer the purpose. This hooping in such a case should not be considered as enabling the concrete to carry any more load.

EXAMPLES FOR PRACTICE

1. A column carries a concentric load of 10 tons and an eccentric load of 30 tons. The distance from the eccentric load to the center of the column is 3 inches. The column is 28 inches square and is reinforced by four $1\frac{3}{4}$-inch square bars. One of these bars is placed at each corner so that the center of each bar is $2\frac{1}{2}$ inches from the two nearest column faces. Find, approximately, the stresses produced, considering that the outside $1\frac{1}{2}$ inch of the column is simply a fireproofing.

$$\text{Ans.}\begin{cases}\text{No tension}\\ s_c = 145 \text{ lb. per sq. in.}\\ s_s' = 1{,}566 \text{ lb. per sq. in.}\end{cases}$$

2. A column is 20 inches square. It contains four rods $1\frac{1}{2}$ inches square. These rods are placed one at each corner, $2\frac{1}{2}$ inches from each face. It carries a load of 10 tons placed 20 inches from the center of the column. The outside $1\frac{1}{2}$ inches is considered to be effective only as fireproofing. What approximate stresses are produced?

$$\text{Ans.}\begin{cases}\text{Compression in concrete, 302 lb. per sq. in.}\\ \text{Compression in steel, 3,912 lb. per sq. in.}\\ \text{Tension in steel, 5,411 lb. per sq. in.}\end{cases}$$

In columns where the eccentricity is so small as to cause no tension in the concrete, the following approximate method will give probably closer results than the one just given. The same notation is used as with double reinforced beams. Also, h equals the width of the column from face to face after the fireproofing has been deducted, measured in the direction of the eccentric loads. By *transformed section* is meant the column section in which the steel is replaced by n times its area in concrete. First the *centroid* or center of gravity of the transformed column section must be located. If u is the distance from the centroid to the compression face after the fireproofing is removed, then $u=\frac{\frac{h}{2}+n\,p\,d+n\,p'\,d'}{1+n\,p+n\,p'}$. The moment of all the loads, whether in the geometrical center of the column or not, must be taken about the axis through this centroid. Let this moment be called M. Let A_t be the area of the transformed section, and I the moment of inertia of this section. Then $A_t=b\,h+n\,(A+A')$ and $I=\frac{1}{3}\,b(u^3+(h-u)^3]+n\,A\ \ (d-u)^2+n\,A'\ \ (u-d')^2$. The total maximum compression in the concrete is $s_c=\frac{W}{A_t}+\frac{M\,u}{I}$ where W is the total load. When $\frac{M\,u}{I}$ is greater than $\frac{W}{A_t}$ tension exists and the previous method must be employed.

As an example, assume a column with the fireproofing coat removed. It is then 21 inches square and carries 8 square inches of steel 1 inch from the face farthest from the eccentric load and 2 square inches of steel 1 inch from the other face. It carries 20,000 pounds placed in the center of the column, that is, 10½ inches from each face, and 10,000 pounds 3 inches nearer the face where the 2 square inches of steel are located. Find the stresses produced. In this case $n=15$, $A=8$, $A'=2$, $d'=1$, $d=20$, $h=21$, and $b=21$. Then $p=\frac{A}{b\,h}$, not $\frac{A}{b\,d}$, as in beams, since the entire section is considered. $p=\frac{8}{21\times 21}$

$=.01814059$ and $p'=\dfrac{2}{21\times 21}=.00453515$. Therefore,

$$u=\frac{\frac{21}{2}+15\times .01814059\times 20+15\times .00453515\times 1}{1+15\times .01814059+15\times .00453515}=11.947 \text{ in.}$$

20,000 pounds is $10\frac{1}{2}$ inches from the compression face. Its moment arm is therefore $11.947-10.5=1.447$ inches. The other load is $10\frac{1}{2}-3=7\frac{1}{2}$ inches from the compression face. Its arm is therefore $11.947-7.5=4.447$ inches. The value of M is therefore $1.447\times 20{,}000+4.447\times 10{,}000=73{,}410$ inch pounds. Then $A_t=21\times 21+15\ (8+2)=591$. Also, $I=\frac{1}{3}\times 21\,[11.947^3+(21-11.947)^3]+15\times 8\ \ (20-11.947)^2+15\times 2\ (11.947-1)^2=17130.12+7783.30+3595.10=28508.52$. Therefore, $s_c=\dfrac{30{,}000}{591}+\dfrac{73{,}410\times 11.947}{28508.52}=50.76+30.76=81.52$ pounds. As 50.76 is greater than 30.76, no tension exists. Since the entire column is in compression, the steel will always be safe. The above method, as was stated, is only applicable to cases where there is no tension and is in that case even only approximate, so low stresses should be used.

REINFORCED-CONCRETE BUILDINGS

(PART 1)

INTRODUCTION

1. The rapid advance made in reinforced-concrete building construction in the last decade is due: (1) to the demand for an entirely fireproof structure at a moderate cost; (2) to the large supplies of domestic cement of excellent quality that can be obtained at a reasonable figure; and (3) to the difficulty of obtaining quickly large timbers of sound material for heavy construction.

Probably the best examples of buildings constructed of reinforced concrete exist in those structures erected primarily for industrial or commercial purposes, such as warehouses and manufactories. For buildings of this kind, reinforced concrete is rapidly displacing the type of construction known as *slow burning*, which has heretofore been extensively employed. Although factories and warehouses undoubtedly furnish the majority of examples of reinforced-concrete construction, yet many prominent and interesting concrete structures in the nature of what might be termed "civil architecture," as well as numerous fine examples of hotels, college buildings, and churches, are in existence.

2. It is possible to divide reinforced-concrete buildings into three distinct classes: (1) those in which the entire structure is of this material; (2) those in which the entire structural framework is of reinforced concrete and the exterior

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faces are of other materials, in order to furnish a better architectural effect than can be obtained by the use of concrete alone; and (3) those in which only the floor systems are of concrete reinforced with steel rods.

The great distinction between the many types of reinforced-concrete construction lies principally in the design of the floors and columns. The design of the other portions of a building does not, as a rule, depend on any one type of construction, but is controlled by the experience and judgment of the designer. The many different types of reinforced-concrete floor systems were fully described in another Section, so that this Section will treat more particularly of the general details of reinforced-concrete buildings.

DETAILS OF CONSTRUCTION

LINTEL AND SPANDREL CONSTRUCTION

3. Influence of Framing Plans.—In the planning of reinforced-concrete building construction, one of the most important features is the design of the lintel construction to be used over window and door openings. If the bays are narrow and the distance between the jambs of the window openings small, so that each beam can have a bearing upon a wall pier, as indicated in Fig. 1, then the slab can span from beam to beam, and practically the only load that comes upon the lintel *a* is that due to the weight of the spandrel wall *b*.

Ordinarily, however, in order to get the greatest amount of window space, or clear daylight opening, in modern commercial buildings, it is necessary to have twin or triple windows and to make the distance between piers from 12 to 16 feet from center to center. If this plan of construction is followed, there are two methods that can be employed. These methods are indicated in Fig. 2.

4. In the construction shown in (*a*), one-half of the floor load from the beams *a* and *b* is concentrated at two points upon the lintel, and the lintel has this load, as well as the weight of the spandrel wall, to sustain. Also, as the beams *a* and *b* extend into the lintel, it must at least be equal in depth to the depth of these beams. Thus, the height of the window opening is materially reduced, and an important percentage of its lighting and ventilating efficiency is consequently destroyed.

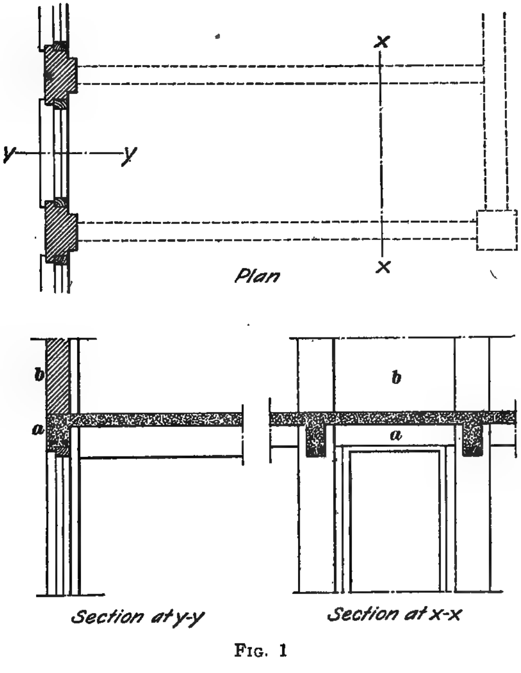


FIG. 1

In order to obtain the greatest amount of window surface with reference to piers and lintel construction, the method of framing the floor illustrated in Fig. 2 (*b*) is frequently employed. As will be noticed, the girders, instead of extending from column to column, as shown at *c* in view (*a*), extend from column to wall pier; consequently, the beams extend over the window opening in a direction parallel with the lintel.

A study of this figure will show that the slab spans from the beam *a* to the lintel *b*, and that, as a result, the lintel carries one-half of the slab load between these structural members and is required to sustain the weight of the spandrel as well. The advantage gained by this method of construction exists in the fact that there are no beams abutting the lintel. Thus, this member may be reduced in depth to the very minimum for strength required to support the small floor load and the weight of the spandrel wall.

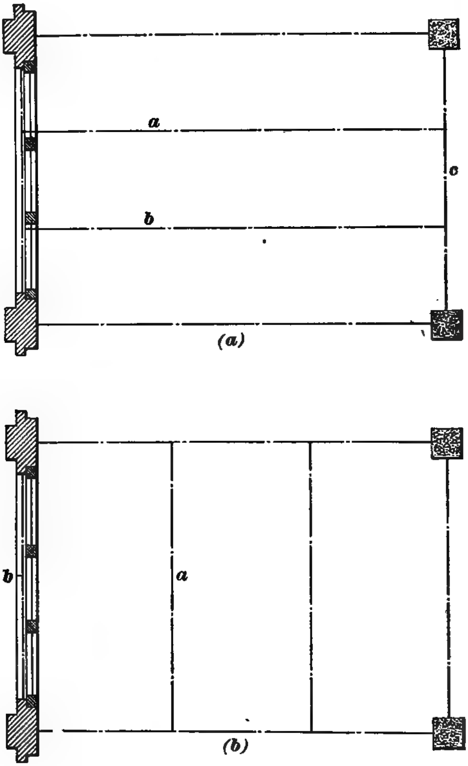


FIG. 2

5. Concrete Lintels for Openings in Brickwork. In order to explain the usual lintel constructions that have been successfully employed in conjunction with reinforced-concrete buildings and floor systems, reference is made to Fig. 3. In this figure is shown the construction usually employed

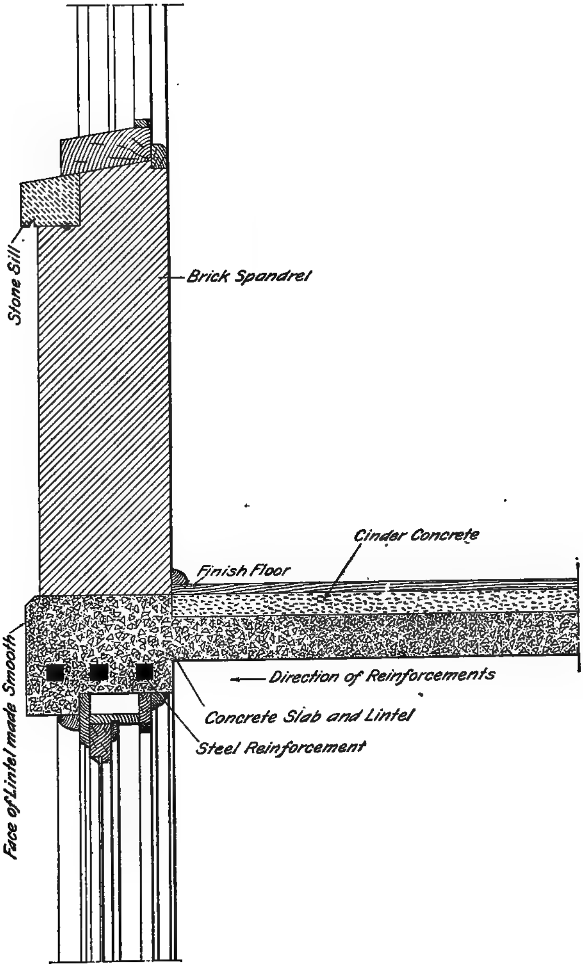


FIG. 3

when the beam framing plan is the same as that shown in Fig. 2 (*b*). The section is taken through the center of the opening.

Frequently, with brick walls and piers, the opening in the brickwork over the window head is arched, as shown in Fig. 4. In this case, the reinforced-concrete slab is made to span from girder to girder, with its reinforcement parallel to the arch span. It therefore needs no support over the arch head, and for this reason the soffit of the arch can come flush with the under side of the slab.

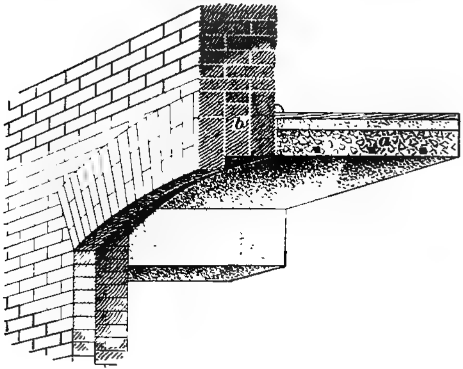


Fig. 4

In Fig. 4 the slab construction is shown at *a* and the brick arch at *b*. Although the soffit of the arch is flush at its highest point with the under surface of the floor slab, it will be observed that at either side of this point some light is cut off by the spandrels of the arch, thus reducing the available daylight opening.

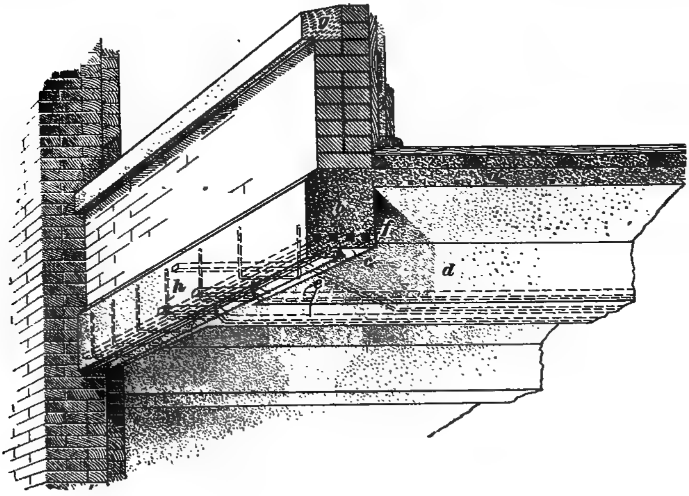


Fig. 5

6. In Fig. 5 is shown a method of forming a concrete lintel over a side window opening where the beams extend from side to side of the building. Here, as in the construction already described, the slab spans in a direction parallel with the wall in which the windows occur, but does not span the entire distance between piers. As shown, the reinforced-concrete slab *a* becomes monolithic with the lintel *b*, which is rabbeted in the soffit at *c* to receive the window frame. The floorbeams, or secondary members of the construction, are shown at *d*, and enter the lintel so that the soffits of the beams are flush with the soffit of the lintel. Where the beams are framed into the lintel in this manner, it is necessary to bend the reinforcing rods *e* of the beams upwards, so that they will have a bearing above the reinforcing rods *f* of the lintels.

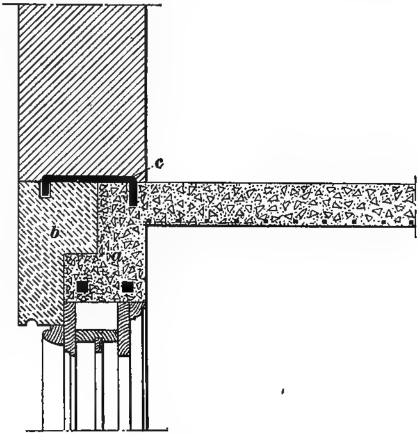


Fig. 6

The spandrel wall in constructions of this kind may be built up of brick, capped with a stone sill *g*, and arranged to receive the window frame. Where the beam rods overlap the lintel rods, it is also advisable to provide stirrups *h* in the lintel, especially at each side of the bearings of the beam reinforcement. In this way, the reinforcement *f* of the lintel bonds with the concrete mass and thus restrains it from pulling away.

7. Owing to the difficulty of obtaining smooth work in placing concrete, exposed lintels should be finished with a bush, or patent, hammer or by brushing the partly set concrete with wire or rattan brushes. The concrete may be smoothed by troweling or finished with cement, but such a finish is liable to peel in time and thus disfigure the work.

If a neat finish is desired on a building that is not too ornate and is constructed with brick piers and spandrels, the lintel and spandrel construction shown in Fig. 6 may be employed. In this figure, the concrete lintel construction *a* with its reinforcement is formed with a ledge of sufficient width to support a cut or sawed stone lintel. By means of the anchor *c*, the stone lintel is further held in place, the ledge formed in the concrete answering as a support for this stone facing.

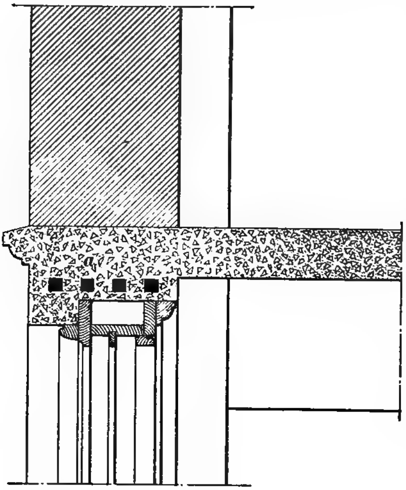


Fig. 7

8. Concrete Lintels and Spandrels.—In buildings of an industrial nature, such as factories and workshops, it is sometimes desirable to obtain a maximum amount of light by running the beams parallel with the lintels, as explained in conjunction with Fig. 2 (*b*). In order that the window head may be as near the bottom of the floor slab as possible, the construction shown in Fig. 7 is commonly employed. Here, the lintel *a* is rabbeted to receive the window head, and it drops down below the bottom of the floor slab a total distance of only 2 or 3 inches. When well reinforced, a concrete lintel of this kind is usually sufficiently strong to carry the load of a portion of the floor slab and the weight of the spandrel. However, it will be found to be too weak where the distance between piers is greater than 8 or 9 feet, and where the floor loads become heavy on account of the assumption of a considerable live load.

In order, therefore, to obtain the necessary strength in this structural member, the lintel beam is increased in depth, being extended above the floor slab, as shown in Fig. 8. A lintel constructed in this manner loses the advantage gained by having the adjoining part of the slab act in compression at the upper section of the lintel. In the present arrangement, the slab section occupies a position near the neutral axis, and its resistance to compression is lost. The lintel must therefore have its section extended upwards to an extent sufficient to give it the needed compressive strength. Some difficulty is encountered in forming a lintel of this kind if the material is very plastic, because such material will ooze out from underneath the form board that forms the projection of the lintel above the slab. Such difficulty, however, can be avoided by using a semidry mixture.

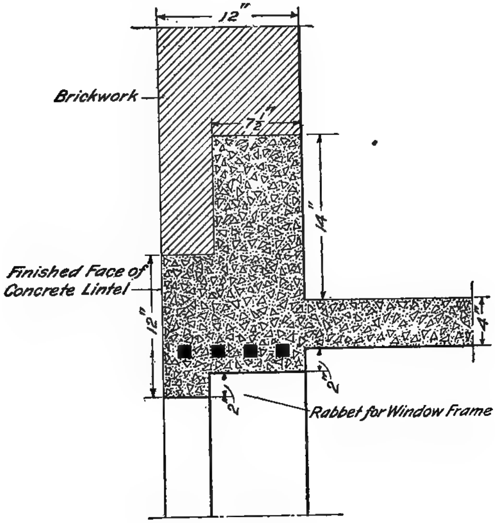


Fig. 8

9. If the span between piers is great and the floor loads heavy, as is usually the case in warehouse construction, it may be necessary, where the soffit of the lintel is brought close to the under side of the slab, to form the entire spandrel of reinforced concrete, as shown in Fig. 9 at (*a*).

In designing construction of this kind, it will be very difficult to form the spandrel monolithic with the floor slab and lintel. Therefore, in considering the constructive features, if it is proposed to use the depth of the entire spandrel to carry the load, the work should be arranged as shown. Here, the lintel is designed so that any junction with the panel of the spandrel may be made on the line *a a*, and the spandrel constructed afterwards. The junction of the work is hidden by the projection of the lintel beyond the face of the spandrel, and the projection finishes on top with a bevel.

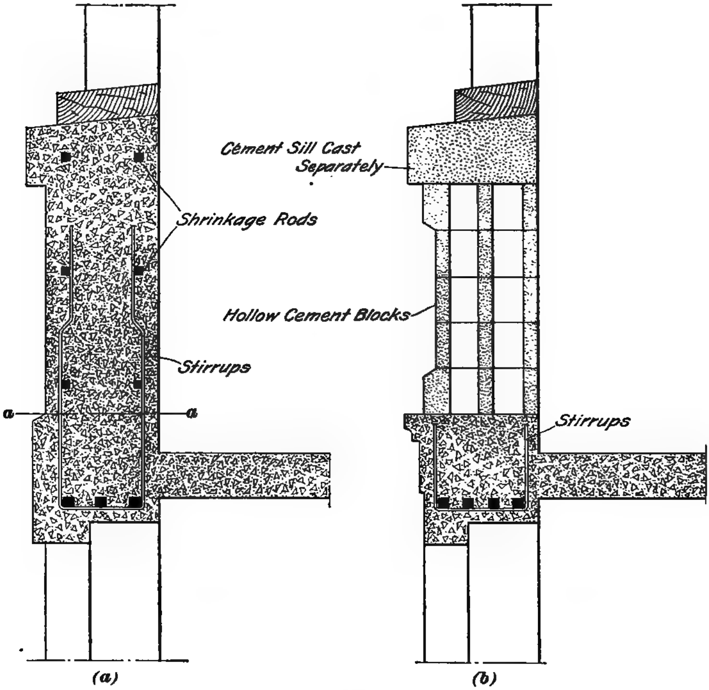


FIG. 9

When this construction is used, either the lintel can be figured as a reinforced-concrete rectangular beam, without including the main portion of the spandrel above the line *a a* and reinforcing the spandrel with sufficient metal to sustain only its own weight, or else long stirrups may be used that will project up into the spandrel and bond the lintel with it in such an efficient manner that they will act as one rectangular beam.

10. In Fig. 9 (*b*) is shown a reinforced-concrete lintel and spandrel construction that can be used on a building that is more or less ornamental. In this construction, the lintel is formed of carefully molded reinforced concrete that is hammer-dressed on the face after it has set. The spandrel wall is formed of molded hollow concrete blocks, and the window sill of either solid molded concrete blocks or artificial stone.

11. Concrete Lintels With Terra-Cotta Faces. Terra-cotta facings are used in conjunction with reinforced-concrete lintels where a building is to have an architectural finish. Some difficulty, however, is met in placing the terra cotta in conjunction with the reinforced-concrete work. The most secure method is to place the terra cotta, laid up to true alinement and in its proper position, on centering and allow it to act as a portion of the forms for molding the lintel. In this way, the concrete enters the spaces between the webs of the terra-cotta lintel and secures it beyond any possibility of displacement.

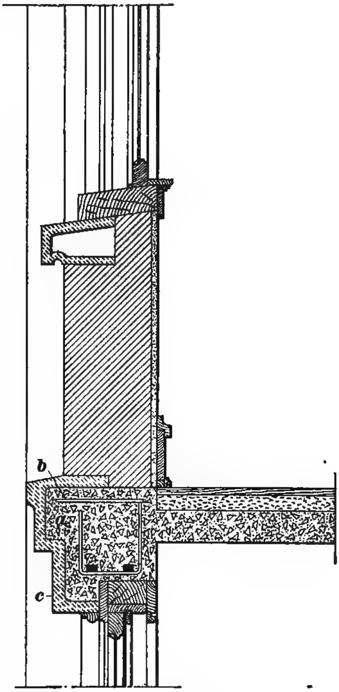


Fig. 10

In constructing the work in this manner, however, the hydrostatic pressure of the wet concrete is very liable to throw the terra-cotta blocks that form the lintel out of alinement; also, if a wet mixture is used, the cement drippings will run through the joints and stain the face of the work. The first difficulty mentioned can be obviated by carefully placing the terra-cotta blocks and then wedging and holding them in place, and the leakage of the wet concrete through the joints can be prevented by carefully pointing the work before the concrete is poured. In practice, it is usually found that the terra cotta, which takes from 5 to 6 weeks to prepare, is not ready when wanted, so that it could not be set with the casting of the concrete even if such a method were to be followed. It is better, therefore, to design the work so that the terra-cotta facing of the lintels can be placed after the reinforced-concrete work has been molded and set.

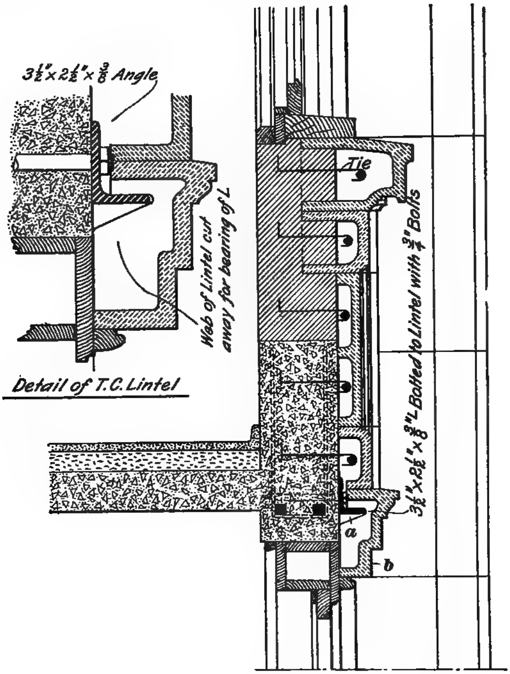

3½"x2½"x⅜ Angle
Web of Lintel cut away for bearing of L
Detail of T.C. Lintel
Tie
3½"x2½"x⅜"L Bolted to Lintel with ¾" Bolts
a
b

FIG. 11

12. One of the best methods to employ in such work is to arrange the concrete lintel as shown in Fig. 10; that is, to provide a projection *a* for the support of the terra-cotta lintel facing. In order to strengthen the projection and prevent it from shearing off, the shear rods *b* are introduced. When the terra-cotta lintel *c* is properly bedded upon the projection, the brickwork of the spandrel will hold it securely in place and the back of the lintel will finish against the wooden frame. One objection to this construction is that the transverse webs of the terra cotta are cut away, thus tending to weaken the work.

13. Another method of supporting terra-cotta lintels in conjunction with reinforced-concrete work is shown in Fig. 11. Here, instead of a projection on the lintel, a steel angle *a* extends along the face of this member. This angle is secured at intervals of 2 or 3 feet by means of bolts that are properly embedded in the concrete work. By using the angle a truer and more level bed is more likely to be obtained for the bearing of the lintel facing *b*, and, besides, more of the transverse webs of the terra-cotta work are retained.

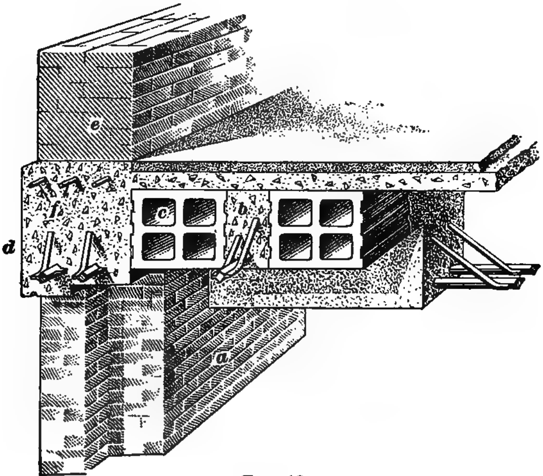


Fig. 12

14. Lintel Construction With Composite Floor Construction.—If the floor construction consists of reinforced-concrete joists with hollow terra-cotta blocks between, it is necessary to use a deep reinforced-concrete lintel over the window openings, as shown in Fig. 12. The construction illustrated shows a composite floor system in which the concrete is reinforced with Kahn bars. The wall piers *a* support girders that run crosswise, the tile *c* and reinforced-concrete joists *b* extending between the girders. The tile is placed next to the concrete lintel *d*, which, owing to the depth of the floor construction and the fact that the piers are about 14 feet from center to center, is made of considerable depth and extends a little above the floor construction. As will be observed, the lintel beam is reinforced with two main reinforcing bars at the bottom and is provided with rods *f* in the upper part at the pier bearings to resist negative bending moment. The lintel projects about 2 inches beyond the face of the pier and supports a brick spandrel wall *e*; it is also rabbeted in the soffit to receive the sash frame.

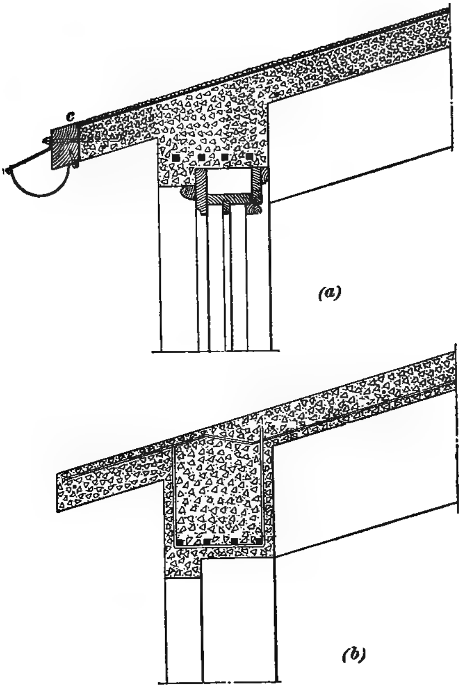


Fig. 13

CONSTRUCTION OF EAVES AND CORNICES

15. Construction of Overhanging Eaves in Concrete.—There are several ways of arranging the construction of monolithic buildings along the eaves of the roof, and there is no material that lends itself so readily to good construction at this point as concrete.

If a manufactory or warehouse is to be erected in a country or a suburban district at a minimum cost, the simplest way to arrange the roof at the eave line so as to take care of the rainwater will be to follow the method shown in Fig. 13 (*a*). Here, the roof slab is projected beyond the wall line, and a copper or galvanized-iron gutter supported by hangers conducts the rainwater from the edge of the roof to outside conductor pipes. This means of catching the drip from the roof is often omitted, as shown at (*b*), and it is not unusual, especially where there is no basement and where the floor level is somewhat above grade, to allow the drip from the roof to drain directly upon the ground.

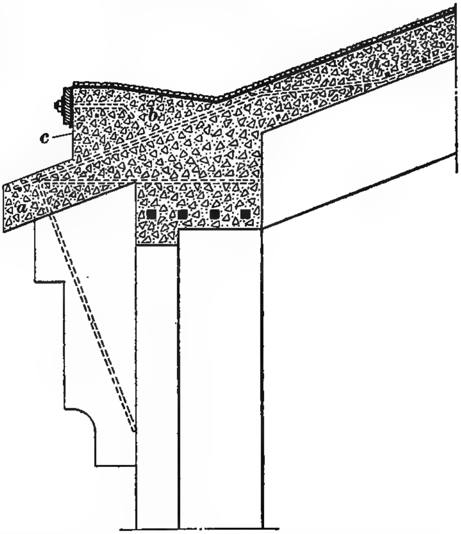


FIG. 14

Where the slab spans between beams or girders that extend from the wall piers across the building, the lintel can be made of minimum depth, as in (*a*), and the window frame brought close up underneath the roof slab. Where the overhang of the roof is considerable, and the slab spans to the lintel, the rods should be bent up as indicated in (*b*), in order to provide the necessary resistance to the cantilever effect of the overhang of the slab.

In this construction, the custom is to employ a slag or gravel roof and to secure this roof under a cleat *c*, Fig. 13 (*a*), bolted against the edge of the concrete slab by means of bolts that are embedded in the concrete work as it is placed. As slag will adhere to steeper slopes than will gravel, a slag covering should always be used on roofs whose slopes exceed a rise of 3 inches to the running foot.

16. Formation of Gutters in Concrete Roof Slab. A more permanent and durable construction than the foregoing for the eaves of a reinforced-concrete building is shown in Fig. 14. Here, the roof slab *a* is extended, being supported in some instances by brackets to provide an ornamental feature. A gusset, and consequently a gutter, is formed by raising the concrete work above the roof, as shown at *b*. The rainwater is thus drained to inside cast-iron conductors, an eave box being formed in the concrete roof slab or provided by embedding in the concrete work a cast-iron box in the nature of a cesspool. The roofing material, usually of slag, is carried down the slope of the roof, across the gutter, and turned down the vertical side *c*. At this point it is secured to a wooden strip covered with galvanized iron. This strip is held in place by bolts previously embedded in the concrete work.

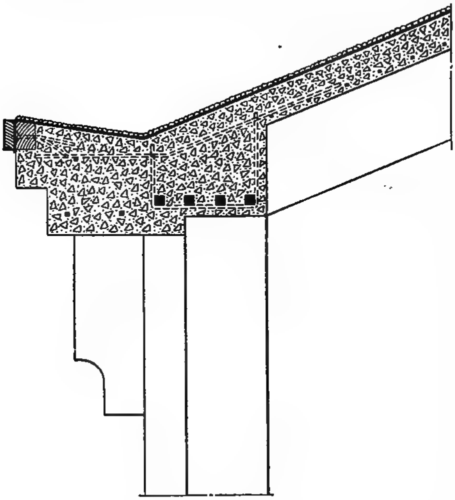


Fig. 15

17. Another method of forming a gutter is to keep all the work adjacent to the gutter below the roof slab by depressing the eaves in the manner shown in Fig. 15. This construction has no particular advantage over the one just described. It is more difficult to place and secure the roof covering, though this can be done, by using a cleat as shown, or by flashing the gutter with metal and running the flashing up under the felt. This last method is not good, because there is no secure way of fastening the felt other than cementing it to the flashing.

18. Construction at Eaves and Parapet Walls.—In monolithic buildings of a more finished nature than those already described, where there is pretension of a cornice formed in the concrete, it is customary to finish the structure at the eave line by running up a monolithic parapet wall, as shown in Fig. 16. The work should be so arranged that the portion below the dotted line *a a*, which is practically a continuation of the roof slab, can be put in at one time and the parapet wall constructed subsequently. This type of construction provides a guard around the roof, supposedly adding security from fire and is frequently required by city and fire-insurance laws, though in monolithic buildings such a parapet wall offers no additional protection.

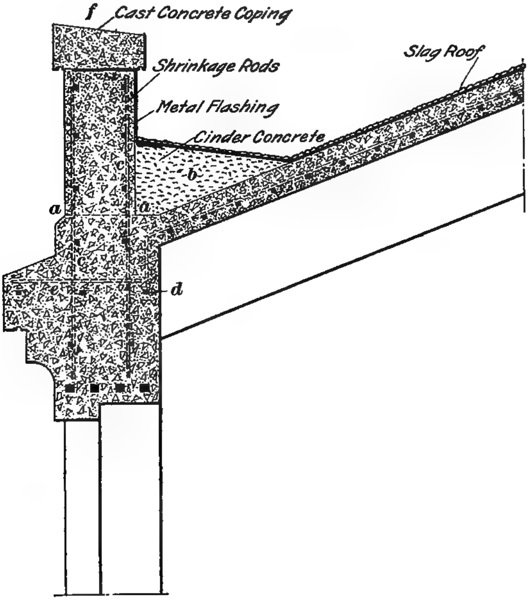


FIG. 16

In forming the gutter in conjunction with this type of construction, the gusset at *b* is made of cinder concrete. This gutter should be properly graded to drain to eave boxes connected to inside conductor pipes.

19. The best way to secure the roofing material in conjunction with a parapet wall of this kind is to place upon the top of the wall a concrete coping *f*. This coping should either be formed monolithic or of artificial stone or concrete blocks previously cast. The metal flashing is bent over the top of the parapet wall and caught underneath the coping; it is also carried down and under the felt, which forms the gutter of the roof. In some instances where it is desired to save the cost of the flashing, which should, especially in all concrete buildings, be of copper, the felt is carried up the vertical face of the wall and turned in under the coping. Gravel or slag, however, will not adhere to the vertical surface of the felt; consequently, the felt, being deprived of this protection, is liable to be torn, damaged, broken, or destroyed by the elements. However, if the cinder filling that forms the gusset is carried to the top of the parapet wall on an easy slope, as shown in Fig. 17, metal flashings may be dispensed with and the felt carried directly under the coping and covered with slag or gravel.

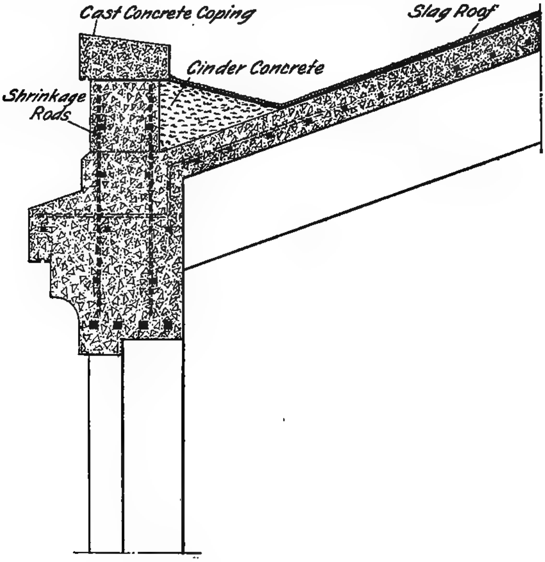


FIG. 17

In this construction, ample shrinkage rods should extend through the parapet wall, and the slab rods should be protected in such a manner as to reinforce any projecting corners or brackets. In Fig. 16, vertical and horizontal shrinkage rods are shown respectively at *c* and *d*, and the projecting rods for the reinforcement of the overhanging portion of the eaves are shown at *e*.

20. Monolithic Cornice Construction.—Buildings constructed entirely of reinforced concrete are frequently finished with an elaborately molded cornice at the top of the main façade. A cornice of this kind sometimes extends above the roof 5 or 6 feet and has an overhang of 4 or 5 feet. Special attention must be paid to the design of such a cornice in order to provide a reinforcement that is ample and so bonded in with the other concrete that there will be no possibility of its falling by reason of the weight of the overhanging portion.

A monolithic cornice is shown in Fig. 18 (*a*). The profile of this cornice consists of the crowning member *a*, the mutules *b*, and the frieze, or entablature, *c*. If the roof slopes in the direction indicated in Fig. 18 (*b*), the lines *k* indicate the joists and *l* the direction in which the slab reinforcement rods are laid. When the roof slopes toward or away from the cornice at an angle *a* with the horizontal, as in (*c*), the slab reinforcement rods will run in the direction of the lines *l* and the joists will be in the direction *k*. The lintel *d*, view (*a*), must be so designed as to carry the weight of the entire cornice, and sufficient reinforcing rods, as at *e*, must be placed in the bottom of the lintel. Besides these rods, there should be provided numerous shrinkage rods *f*, usually of $\frac{3}{8}$-inch round or $\frac{5}{16}$-inch square twisted bars, running lengthwise of the cornice. The weight of the overhanging portion must be calculated, and its bending moment about the point *g* obtained. Then rods *h* projecting from the slab can be extended to the top of the projecting portion so as to furnish the necessary resistance; and, if necessary, additional rods *i* extending down through the lintel and hooking to the reinforcing rods in this member can be introduced.

Where mutules, modillions, or brackets occur, it is well to place shrinkage or shear rods, as shown at *j*. The top of the construction may be covered with copper, which should be securely fastened to cleats embedded in the concrete work

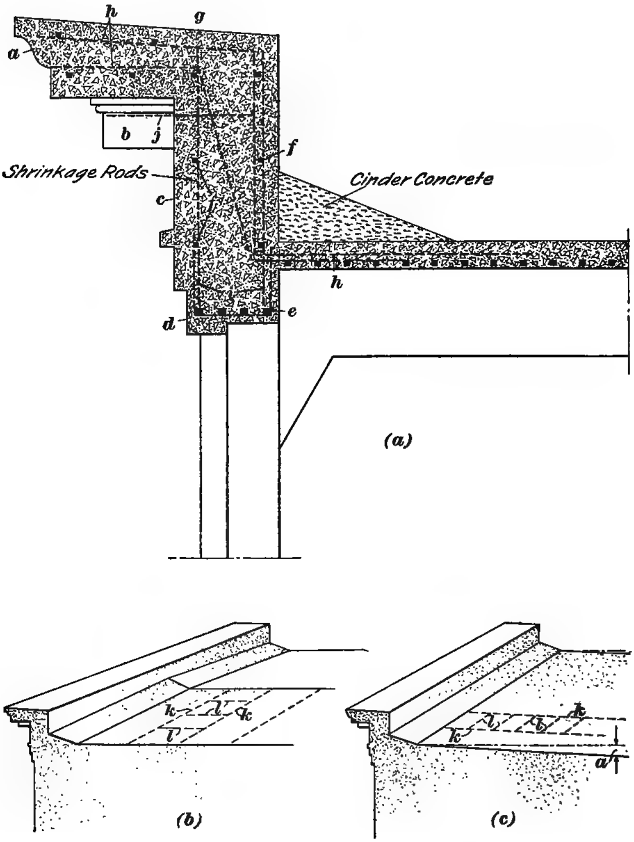


Fig. 18

and carried down so as to make water-tight junction with the roof covering. Sometimes the mutules and the crown molding consist of separate blocks that are molded previously and then built, or set, in the concrete work. By planning the construction in this way, any defect in the finer molded members can be corrected before they are built in; whereas, if the work is cast monolithic, it is difficult to make good any faulty work caused by carelessness or accident.

21. Terra-Cotta Cornice With Reinforced-Concrete Construction.—Unless the terra-cotta work is made a portion of the form and the concrete poured in, so that the terra-cotta facing becomes securely embedded in the monolithic construction, it is only with difficulty that a terra-cotta cornice is properly designed and constructed in conjunction with the reinforced-concrete work.

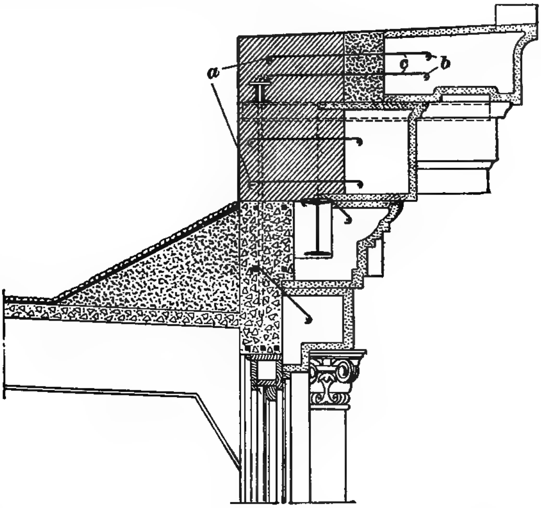


FIG. 19

A familiar type of terra-cotta cornice construction used in conjunction with reinforced-concrete work is shown in Fig. 19. The main overhang of the cornice is upheld by small **I**-beam outlookers. These outlookers are held down at the end by two channel irons that extend the entire length of the cornice. The channels are placed back to back, with anchor bolts between them extending down into the concrete lintel. Rods *a* are embedded in the concrete work, and other rods *b* are inserted through holes in the webs of the terra-cotta blocks. Each pair of rods is connected by wires, or strips, *c*, bent around the rods. The method of supporting the several parts is clearly shown in the figure. The support of each separate block should be carefully studied to see whether it is properly secured in place and whether or not the work can be put together in the field.

BRICK FACING AND TERRA-COTTA STRING-COURSES

22. Fastening of Brick Veneer. — One difficulty encountered in the construction of reinforced-concrete buildings whose exterior walls are to be faced with brick is to arrange the proper means of securing the brick veneer, or facing, to the wall piers or spandrels. If the bricks rest upon a solid ledge of concrete, a stone band, or a base course, and their weight is consequently securely sustained in this manner, they can be tied to the concrete wall at intervals with flat copper ties. These ties are placed through slots in the form boards, usually made by boring two small holes side by side and cutting out between, as shown in Fig. 20.

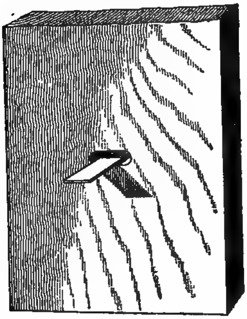


Fig. 20

The method of tying a brick facing to a concrete pier by means of copper or galvanized-iron ties is illustrated in Fig. 21. As shown, the ties, which consist of a strip of copper about $\frac{1}{16}$ inch thick, $\frac{3}{4}$ inch wide, and 7 inches long, are placed every seventh joint. These ties are spaced about 2 feet apart horizontally, and are embedded in the concrete work to a depth of about 4 inches, which gives them, when a $\frac{5}{8}$- or $\frac{3}{4}$-inch space is left between the facing and the concrete, a surface about $2\frac{1}{4}$ inches long in the bricks. It is customary in arranging the brick facing to allow about 5 inches for the width of the brick and the space back of the brick, which is necessary on account of irregularities in the forms. This space, which is shown at *a*, Fig. 21, is usually flushed in with mortar as the bricks are laid.

23. Brick Facing With Terra-Cotta String-Courses. In the architectural treatment of a building constructed with a reinforced-concrete frame, it frequently happens that much of the brickwork bears upon terra-cotta band or string-courses, as illustrated in Fig. 22. If the terra cotta is adequately supported and filled with concrete so as to have great bearing resistance, it will sustain the load of the brickwork without difficulty. Usually, however, such terra-cotta band courses possess little strength and are mostly built and tied in with only sufficient security to support their own weight. It is inadvisable, therefore, to allow the entire weight of the brick facing to come upon such decorative work. The brick facing should consequently be supported by the concrete skeleton frame of the building by means of concrete ledges *a* at each story and at the spandrels; or, ledges *b* can be worked on the concrete piers. The latter ledges should be made the thickness of a brick course, and should be arranged so that they will space out with the bricks. In this way, the bulk of the weight of the brick facing will be transmitted to the concrete. Some difficulty is met in laying up the bricks so as to work out even joints, but it is practicable if the ledges are accurately spaced on the concrete piers. If the terra cotta is not of sufficient strength, probably the best way to support such brick facing is to design the terra-cotta band course with an upper member covering a reinforced-concrete ledge capable of sustaining the weight of the brick facing.

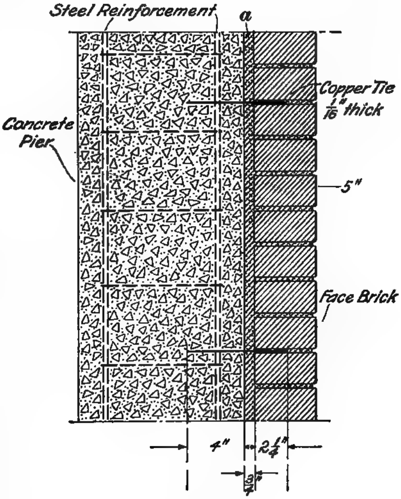


FIG. 21

FASTENINGS IN CONCRETE

24. Reinforced-concrete construction has two disadvantages. The first one is that reinforced concrete is entirely unsuitable for temporary structures, because it is only with great difficulty that a structure erected in this material can be torn down, or demolished, and when demolished there is no salvage to be gained, as the difficulty of reclaiming the steel from the concrete would cost more in labor than the scrap material would be worth. The other disadvantage consists in the difficulty encountered in drilling into reinforced concrete and of securing fastenings for equipment, shafting, or other installation.

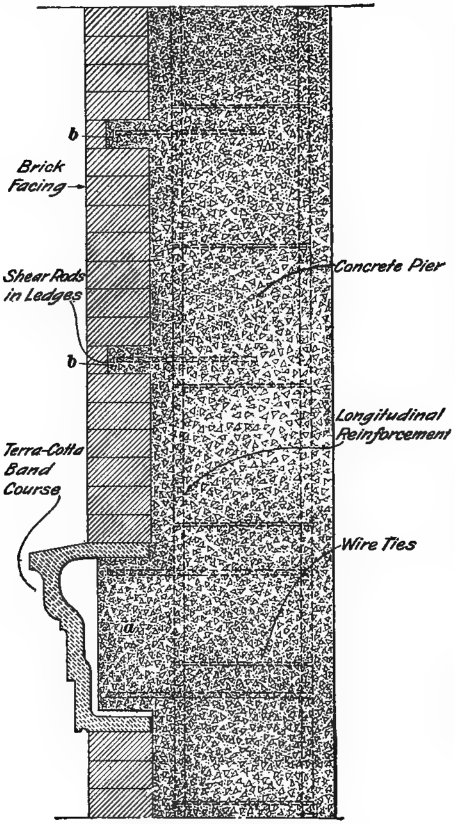


Fig. 22

There are numerous devices intended to be embedded in the concrete work for the purpose of holding **T**-headed or tap bolts in a secure manner, so as to realize the full strength of the bolts. These necessarily must be arranged in the forms before the concrete is poured. Many forms of sockets have been used from time to time, and the ingenuity of the designer can be displayed in the adaption of the material at hand, or that most readily obtainable for the construction of these sockets or fastenings.

25. Size and Spacing of Sockets or Fastenings in Concrete.—In nearly all large commercial or manufacturing buildings, it is necessary to fasten equipment or machinery to the ceiling, and as the cost of the sockets is comparatively small, a sufficient number of them should be provided in all beams or girders to answer any purpose. Ordinarily, they should be tapped out for ¾-inch bolts, but where heavy machinery or hoisting apparatus is to be employed, it will be better to use ⅞-inch tappings in the sockets and to provide ⅞-inch anchor bolts. Sockets or other fastenings should be provided near the end bearings of each beam and girder, and it is well to space them intermediately, not farther apart than 4 or 5 feet.

A ¾-inch bolt will ordinarily test in a tensile pull to from 15,000 to 17,000 pounds. Thus, if each socket has sufficient bond in the concrete, or a good grip, or hold, upon the reinforcement, it can be considered to sustain safely a load of about 4,000 pounds.

Where it is necessary to install a heavy main, or head, drive from a large engine or power generator, or to bolt heavy motors to the ceiling, such sockets should not be employed. In such cases, the location of the head drive or the motor should be ascertained, and heavy through bolts should then be embedded in the concrete work in order to provide against failure from the heavy loads or pulls caused by such machinery.

26. The Unit Socket.—The **Unit socket,** which is the salient feature of the unit system of girder-frame construction described in a preceding Section, is one of the best devices for bolting shafting and securing piping or other equipment to the soffit of beams or girders. This socket, a detail of which is shown in Fig. 23, is particularly efficient from the fact that it grips the steel reinforcement and cannot be pulled from the concrete without tearing away the reinforcing metal, which is securely bonded in the monolithic beam or girder by means of stirrups and its own bond.

The Unit socket shown is arranged for a narrow reinforced-concrete beam in which the four reinforcing bars are placed in pairs, one above the other. The socket *a*, which may be of either cast steel or malleable iron, is tapped out for a ¾-inch tap bolt *b*, by means of which the socket and the steel reinforcement to which it is attached by the bolt *c* are held in a correct position in the forms.

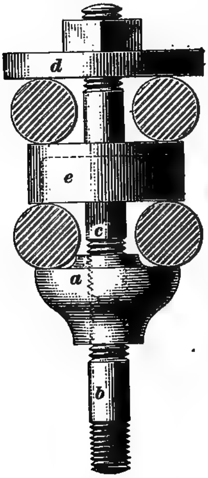


FIG. 23

The lower part of the bolt *b* projects through a hole in the form bottom, to which it is temporarily clamped by means of a nut that is screwed up on the lower end. The bolt *c* is sufficiently long to extend between the reinforcing rods or bars and, by means of the heavy plate washer *d* and the casting *e*, clamps the rods of the girder frame. Thus, besides assisting in holding the rods together, this bolt secures the socket *a* to the steel reinforcement in such a manner as to prevent it from pulling away from the construction, making the weakest point of the device at the root of the ¾-inch threaded bolt.

27. Malleable-Iron and Cast-Steel Sockets.—There are many other types of sockets; some of them have been patented, and others are merely expedients that have been designed from time to time to meet the requirements of building construction. In Fig. 24 are shown two forms of malleable-iron or cast-steel sockets. The socket shown in view (*a*) consists of a pronged, or forked, casting tapped out for the bolt required, as indicated at *a*. The prongs are cast with a turn at the ends to provide additional bond or grip in the concrete, and, when the socket is placed at the center of the beam or girder, this form is admirably adapted to reinforcement consisting of three rods, because the center rod can lay on the fork. These sockets are secured to the form board by means of the small lugs *b*. These lugs are notched to receive nails, which are driven into the bottom form board.

The socket shown in (*b*) is somewhat similar in design to the forked socket. It is made of either cast steel or malleable iron, its form being efficient to secure a good hold in the concrete. This type of socket can have lugs cast on the side, or it can be carried up to a height sufficient to allow it to be braced at the top of the slab forms, in which case the lower end is held in place by driving a large nail into the form board and allowing the hole tapped out for the bolt to slip over it.

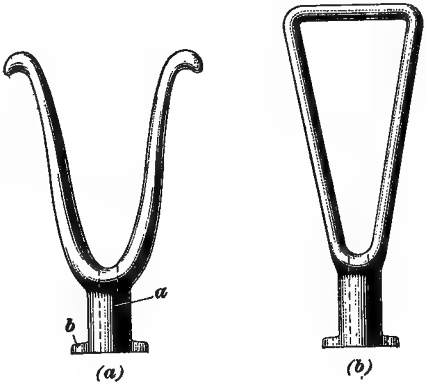
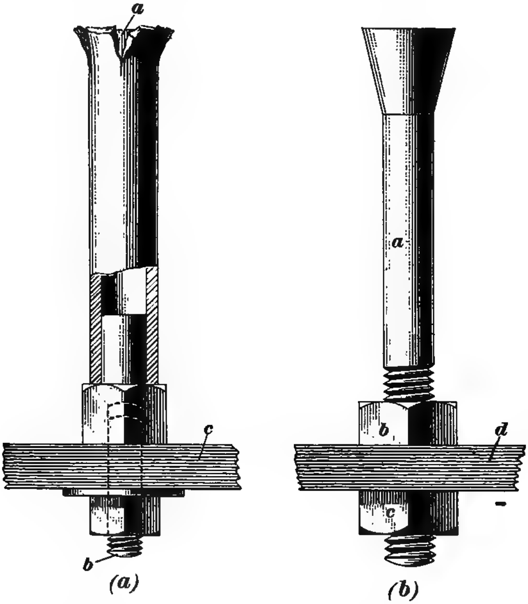


FIG. 24

28. Pipe Sockets and Bolt Fastenings. Two more types of socket are illustrated in Fig. 25. The socket shown in (*a*) is known as the **Jennings-Steinmetz socket.** It consists of a piece of pipe swaged out or broken at the upper end, as indicated at *a*, with a solid wrought-iron bar or block welded to the other end, the solid portion of the block being tapped out for the bolt *b*, which passes through the bottom board *c* of the form.

The socket shown in (*b*) can be used where there is no objection to the bolt end extending below the soffit of the beam or girder. It consists of a rod *a* threaded at one end and upset at the other. At the lower end it is provided with a nut *b*, which will form a bearing for placing the bolt in the forms. By screwing up the nut *b* on the threaded end and passing the latter through a hole in the bottom form board *d*, a nut *c* may be screwed on the projecting end, as shown. In this manner, the bolt can be secured to the form and held there rigidly while the concrete is being placed. The upper end can be upset, split, and swaged out, or, if the beam or girder is of considerable depth, this end can be left plain, the bond in the concrete being sufficient to realize the full strength of the net section of the bolt.

29. It is frequently desirable in reinforced-concrete construction to provide means by which through bolts can be placed in beams, girders, or columns. If the position these bolts are to occupy can be determined beforehand, they may be placed in the forms and embedded in the concrete, in which case the threaded ends should project. This practice, however, is not so good as to place in the forms pieces of pipe having an inside diameter great enough to allow the bolts to pass through. Such pieces of pipe are more conveniently placed in the forms than are bolts, and, besides, they provide a neat, steel-lined hole in the concrete through which the bolts may be slid in and out with little friction.



(a) (b)

Fig. 25

30. If it is found necessary to provide bolt holes after the concrete has been placed, they can be drilled with a diamond-pointed drill. In all instances, such bolt holes should be located on the plans so as not to cut or damage the reinforcing rods. If rods are encountered, not only is the construction likely to be weakened by the cutting of the steel and the damaging of the concrete, but great difficulty will be met with in making the bolt hole accurate. It is also difficult to place any kind of fastening in the soffit of reinforced-concrete beams unless provision is made in the placing of the concrete work. This is due to the fact that, in drilling into the concrete, the steel reinforcement is encountered at a distance of 1½ to 2 inches from the surface, and, besides, such a fastening must be in the nature of an expansion bolt, which is unreliable when subjected to a pulling stress.

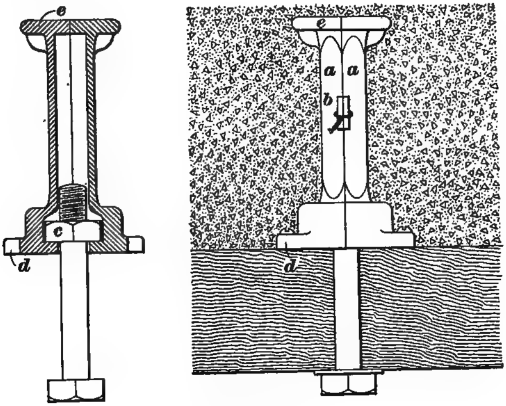


FIG. 26

31. Concrete Inserts.—In Fig. 26 is shown a type of iron insert, or socket, for concrete construction known as the **Hancock insert.** This device consists of a cast- or malleable-iron casing that is made in loose halves *a*, and is wired together through the lugs *b* cast on the sides. Before the two halves of the insert are wired together, however, there is placed in the recess a nut *c*, into which a bolt can be screwed from beneath. The castings are arranged with lugs, or flanges, *d*, on the face end by means of which the insert may be screwed or nailed to the form board, and thus secured in an upright and secure position when the concrete is placed. The flange *e* at the top of the insert furnishes additional bond, or key, with the concrete.

The Hancock insert is made in 3-, 4-, and 6-inch lengths for $\frac{1}{4}$-, $\frac{3}{8}$-, $\frac{3}{4}$-, and 1-inch bolts. The bolts are capable of considerable adjustment, on account of the play, or allowance made for the nut in the recess of the casting. This play is always an advantage when shafting, machinery, or other apparatus of this character is to be installed.

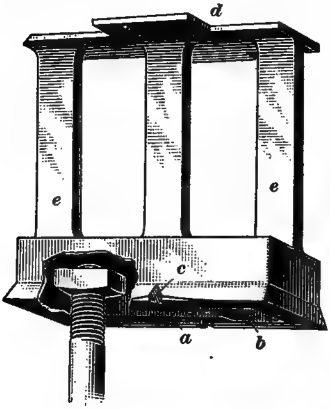


FIG. 27

32. Adjustable Concrete Sockets.—In Fig. 27 is shown an adjustable concrete socket known as the **Bigelow socket.** This casting, which is made of malleable iron, is arranged with a slot *a* that is enlarged at *b* so as to receive the head of the bolt. In order that the casting shall be securely held in the concrete work, it is arranged with a plate *d* and connecting pieces *e*. On the sides of the socket at the face are cast notches *c* into which nails may be driven so that the device may be secured to the forms or centering.

The Bigelow socket is made for $\frac{1}{4}$-, $\frac{5}{16}$-, $\frac{3}{8}$-, $\frac{7}{16}$-, $\frac{1}{2}$-, and $\frac{3}{4}$-inch bolts. The smaller sizes are suitable for embedment in slab work, and the $\frac{7}{16}$-, $\frac{1}{2}$-, and $\frac{3}{4}$-inch sizes may be used in beams and girders.

Three methods of securing this type of socket to the forms are shown in Fig. 28. In the method shown in (*a*), the socket is secured by nailing through the lugs on the side of the casting; in (*b*), a bolt with its head in the slot and passing through the form board is used; and in (*c*) a bar-iron strap, or stirrup, is passed over the top of the casting and its ends are secured with nails to the form boards.

The advantages claimed for these sockets are that they are in one piece, can be furnished at a reasonable figure, and permit of considerable adjustment in the bolt, the bolt being secured in any position necessary by means of a locknut, as shown in Fig. 29.

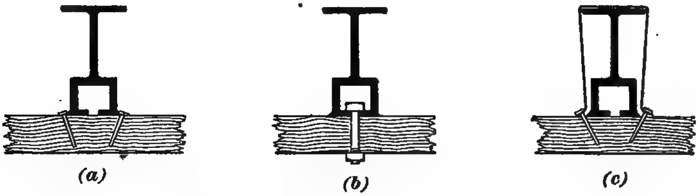

(a) (b) (c)

FIG. 28

33. Sheet-Metal or Flat-Iron Sockets.—The socket illustrated in Fig. 30 (*a*) shows how heavy sheet-metal or thin bar-iron strips may be used as concrete sockets, or inserts. The device shown consists of a piece of heavy sheet metal or bar iron that is cut out on the sides, as shown at *a*, and bent into a **U**, or stirrup, form. In the bottom of the stirrup is punched a hole sufficiently large to allow the bolt that is to be used with the insert to pass through. At *b* is inserted an ordinary wrought-iron square nut, and above this nut is placed a sheet-iron shield *d* to keep the hole in the nut from filling with concrete. This shield has extensions with holes *e*, as shown in (*b*), through which may be driven nails that will fasten it to the bottom board of the form. The sheet *d* and the enclosed nut are held in place by the corners *c*, which are bent or hammered inwards. The notches *a* are so arranged in the sides of the stirrup that the reinforcing rods *f* may pass within them and the insert in this way be more firmly embedded. In the top of the stirrup may be punched holes *g*, through which a round rod *h* may be passed. The slab rods may be wired to this rod, or, if the socket is turned the other way, the slab rods may be passed directly through it.

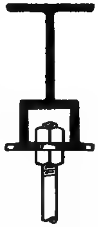


FIG. 29

34. Continuous Inserts for T-Headed Bolts.—In the construction of factory buildings it is sometimes desirable to arrange a slot in the bottom of the concrete beams and girders so that a **T**-headed bolt can be introduced at any point along the entire length. To accomplish this, several methods are employed, the principal ones being illustrated in Fig. 31.

In (*a*) is shown one method of securing an adjustable bolt to the soffit of a concrete beam or girder. The bolt *a* has a **T**-shaped head and is suspended between two angle irons *b*, as shown. The bolt is inserted by slipping the **T** head lengthwise in the slot and then turning it, when in the desired position, at right angles, as indicated. The two angle irons are secured to the concrete by the bolt *d* with its accompanying yoke, as shown in (*b*). The slot *c* in which the **T** bolts slide is formed by placing a plank in the mold when casting the beam.

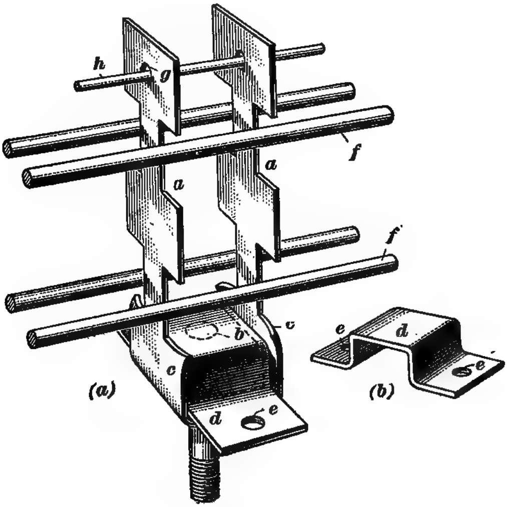
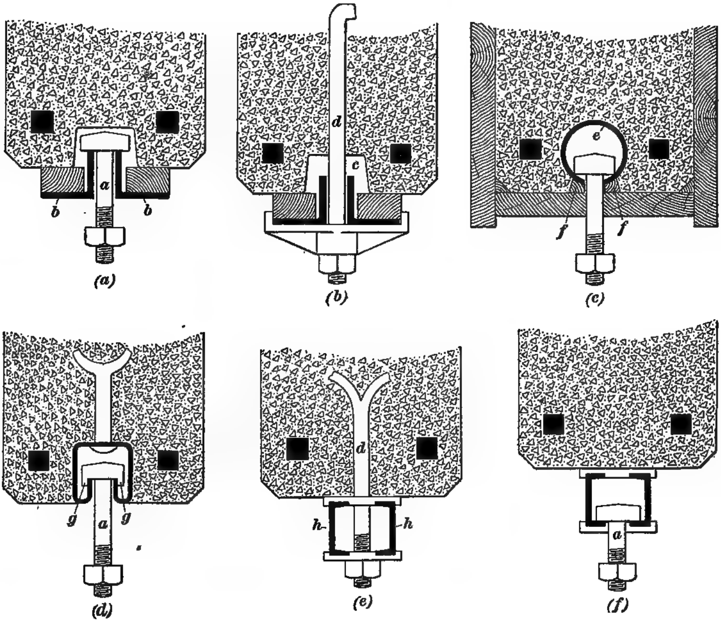


Fig. 30

Another method of arranging for adjustable bolts is shown in (*c*). It consists in inserting a pipe *e* near the soffit of the concrete beam. This pipe is split and forced apart so as to receive a **T**-headed bolt, as shown. It is held in place in the molds by means of the cleats *f* while the concrete is being poured. A modification of this method is shown in (*d*). Here, a special forged shape, known as the *Coburn track*, is used in place of the split pipe. This form is held in the concrete by means of the branched bolt, as shown. The **T** bolt *a* in this case is of special pattern, being provided with two lugs *g* that fit over the edges of the track.



(*a*) (*b*) (*c*) (*d*) (*e*) (*f*)

FIG. 31

Another device for providing a continuous fastener for bolts in the bottom of concrete beams is shown in (*e*). It consists of two light channel irons *h* held at the requisite distance apart by top and bottom plates, or separators. This channel-iron arrangement is secured to the concrete work by means of the bolts *d*. The **T**-headed bolt *a* in view (*f*) can be inserted through the slot formed by the lower flanges of the channel iron, and when given a half turn can be used for securing shafting or equipment at any position.

Two important points to remember in designing such devices are that they shall have sufficient strength, and that they shall be so arranged that in case of fire they will not transmit heat to the steel reinforcement.

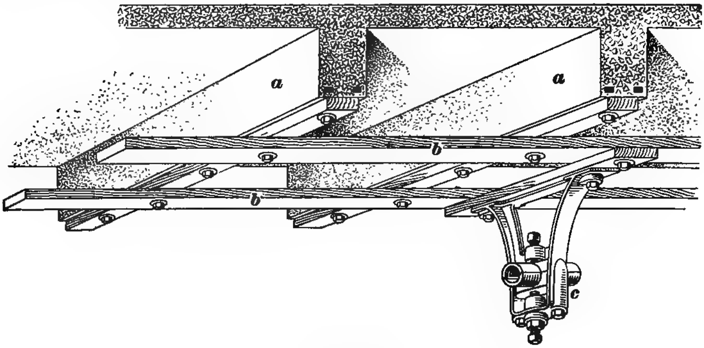


Fig. 32

35. Arrangement of Sockets and Fastenings.—As shown in Fig. 32, the sockets just described may be used to secure the framing required to support shafting. In this case, the sockets in the beams *a* are employed to bolt up the stringer pieces *b*, and they thus provide a firm and rigid support for the hanger *c*. Although girders are usually made somewhat deeper than the beams in reinforced-concrete work, it is frequently preferable to make both sets of members of the same depth, so that their soffits will be on one plane.

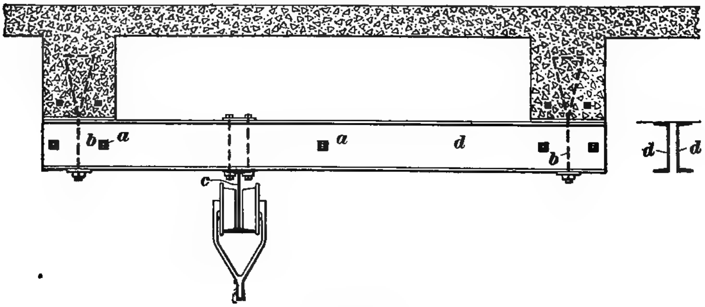
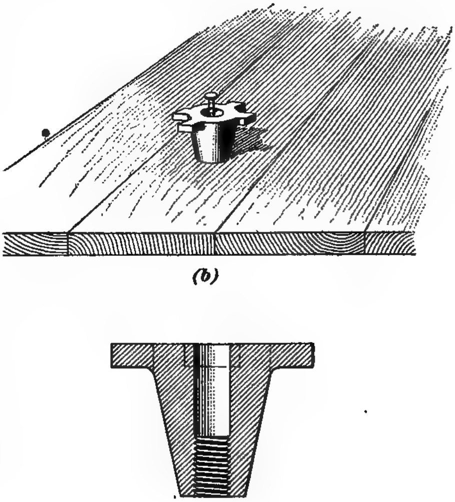


Fig. 33

In this way, the support of shafting and other equipment is greatly facilitated.

In Fig. 33 is shown another use of the sockets. In this case an **I**-beam track for a trolley hoist is supported by two light channel irons *d*. These channels are placed back to back, being spaced by means of bolts and separators *a* and secured to the concrete beam with bolts *b*. The trolley track *c* can be fastened to the channel irons at any point. Channels arranged in this way will securely support curves, frogs, or switches, and are sufficiently strong to carry the usual load placed upon hoists of this character.



(b)

(a)

Fig. 34

36. Sockets for Slabs. — It is good practice in reinforced-concrete work, especially in industrial plants where the electric-light and power wiring is to run upon the soffit of the floor slabs, to embed therein small cast-iron sockets tapped out to ¼ inch. For this purpose, the type of socket shown in cross-section in Fig. 34 (*a*) can be used. Several sockets can be placed at intervals upon the form boards for the slab and they can be held in place with a nail, as shown in view (*b*), while the concrete is being tamped. Usually five or six sockets are sufficient for one panel. Slab sockets must always be of a type that can be securely fastened to the slab centering, because they are subjected to such hard usage in placing and tamping the concrete that they are readily disturbed and displaced, which is liable to render them useless.

FLOOR FINISH FOR REINFORCED-CONCRETE BUILDINGS

37. As the top surface of a reinforced-concrete floor slab is not sufficiently smooth to form a finished floor, it is customary to supply a floor finish of some kind upon the top of this surface. The cheapest finish that can be used is a 1-inch cement-finish coat. This coat, however, will not adhere to the concrete work unless it is put down immediately after the concrete has been placed in the forms, or unless great care is taken to put it on in several successive coats and to work these well into the surface of the slab. With very rough work, the top of the concrete is troweled smooth with about a ¼-inch coat of cement mortar, which is put down at the same time that the concrete is placed.

38. Cement Floor Finish.—The best practice where a cement floor finish is desired is to place on the top of the concrete slab 2 inches of cinder concrete, usually called *cinder-concrete fill*, or *filling*. This concrete is made up of 1 part of cement to 3 parts of sand and 7 parts of clean boiler cinders. The top of the cinder fill is finished with a 1-inch cement-finish coat consisting of 1 part of Portland cement, 1 part of sand, and 1 part of crushed rock. This finish is troweled smooth, indented if required, and marked off in blocks. A section of a floor finished in this way is illustrated in Fig. 35. When constructed in this manner, the finish need not be put down at the same time as the concrete of the reinforced work.

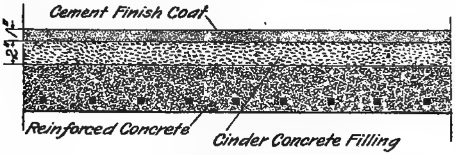


FIG. 35

39. Wooden Floor Finish.—If the finished floor is to be of wood, it is usually constructed in the manner illustrated in Fig. 36. On top of the reinforced-concrete slab are placed beveled sleepers. These are usually made of 2″ × 3″

yellow pine or chestnut, and are placed 16 inches from center to center, extending in a direction at right angles with the flooring. The space between the sleepers is filled with cinder concrete which is well tamped and made almost flush with the top of the sleepers. In commercial and manufacturing buildings, the flooring usually consists of $\frac{4}{4}$ or $\frac{5}{4}$ maple tongued-and-grooved flooring with $2\frac{1}{4}$-inch face. This flooring is secretly nailed to the sleepers and left as laid, or the joints are struck and, in some instances, the floor is planed. Instead of the beveled sleepers, sleepers with straight sides may sometimes be used; these are held in the cinder-concrete fill by means of nails driven in the sides at intervals of from 2 to 3 inches between centers. The sleepers to which a wooden floor finish is secured should never be embedded directly in the reinforced concrete, because such practice would destroy the strength of the work.

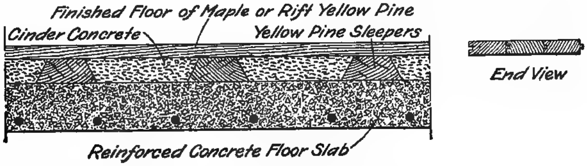
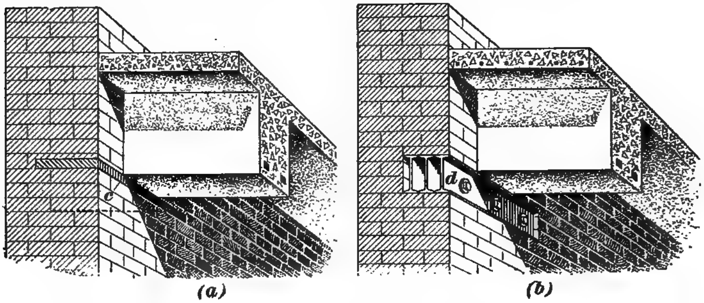

Finished Floor of Maple or Rift Yellow Pine
Cinder Concrete
Yellow Pine Sleepers
End View
Reinforced Concrete Floor Slab

FIG. 36

40. Mosaic, Terrazza, and Plastic Floor Finishes. The reinforced-concrete slab of floor systems may be covered with some other material, such as tile, mosaic, or terrazza, by using a layer of cement about 1 inch thick in which to embed the work. The patented flooring finishes, such as Asbestolith, or Karbolith, may also be used. This kind of flooring consists of a finished coat of plastic material, which is put down in about a $\frac{3}{8}$- or $\frac{1}{2}$-inch thickness. When finished, it has the appearance of plain linoleum, although it is much harder. Such floors as this are used in watch-case and other factories where the sweepings contain valuable material.

BEARINGS FOR CONCRETE BEAMS AND GIRDERS

41. Where concrete beams or girders bear upon masonry walls, the bearings should be investigated to determine whether or not the safe unit pressure upon the masonry is exceeded. In most cases, beams and girders in building construction are from 8 to 12 inches in width, and it frequently happens that the area of the beam bearing is insufficient to transmit the load to the masonry, so that the allowable unit pressure on the masonry is exceeded. The concrete in the beam or girder can be considered to transmit safely 500 pounds to the square inch, which is twice as great as the safe bearing strength of brickwork laid up in cement mortar, and five times that of rubble masonry laid up in cement and lime mortar. The bearing of the beams and girders may therefore be increased by using cast-iron or steel bearing



(a) (b)

FIG. 37

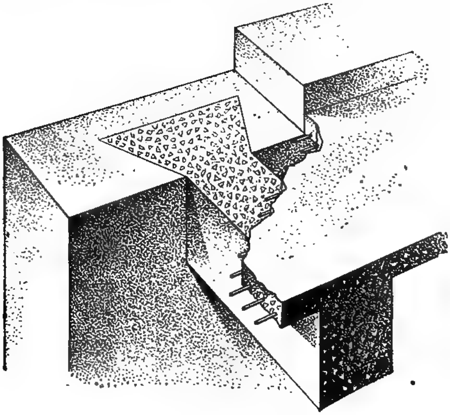


FIG. 38

plates *c* or grillage beams *d*, as shown in Fig. 37 (*a*) and (*b*). Where it is necessary to secure only a small additional bearing area, the end of the beam can be molded with flared sides, as shown in Fig. 38. This device will usually increase the bearing area as much as 50 per cent.

42. Another method sometimes used to increase the bearing area of concrete beams and girders is shown in Fig. 39. In view (*a*), the end of the beam is formed with projecting spurs at right angles to the axis of the beam, these spurs being of such length as to distribute the bearing over a considerable area. If the line *a b* is at an angle of 60° with the horizontal, no steel reinforcement need be used in the cross-beam or bearing lintel; but if this angle is reduced, it is well to use reinforcing rods, as shown at *c*.

In order to save concrete, the bearing of the beams may be increased by the method shown in (*b*). This construction, however, is not so practical as the one shown in (*a*), because the form is more troublesome to make and fill, and, besides, the masonry does not build so readily with flat beds on the slope as it will with the construction shown in (*a*).

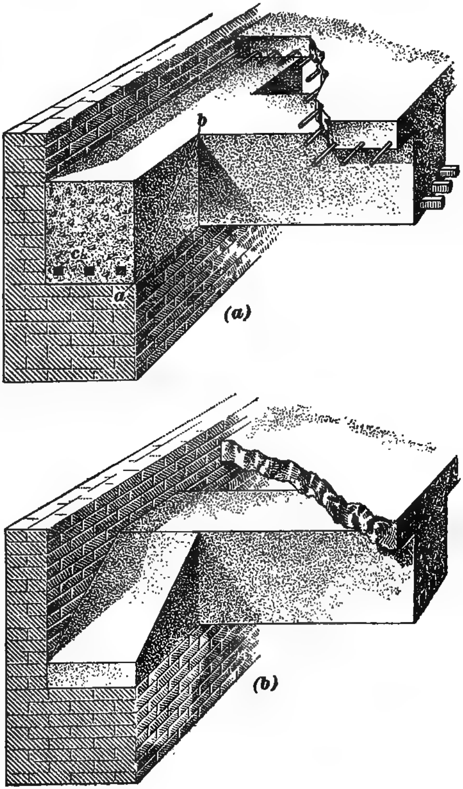


Fig. 39

REINFORCED-CONCRETE STAIR CONSTRUCTION

43. In concrete buildings, it is customary to construct the stairs of reinforced concrete. Although such stairs are more costly than wooden ones, it is convenient, where the operation of reinforced-concrete construction is being carried on, to build the stairs of this material. Wooden stairs, however, are entirely practicable in such a building, as they are always enclosed in a brick or a concrete shaft cut off from the building with tin-lined doors, and consequently would not form a fire menace to the building or its contents.

44. Although the general design of the reinforced-concrete stairs must comply with the requirements of the architectural plan, yet their details of construction are governed by the manner in which it is proposed to construct the building.

If the building is to be of the reinforced-concrete skeleton construction, with the columns and wall piers, together with floor systems, forming a complete structural monolith, the usual practice is to arrange to build the stairs during the construction of the building and to fill the forms of the stairs at the same time that the columns and floor systems are constructed. This method makes the strongest construction, as the stairs become incorporated with the concrete of the building; besides, they may be so arranged that the walls surrounding them to form the stair towers can be built after the entire concrete work has been completed.

In the construction of some buildings, however, where the work is rushed, it is not an unusual practice to construct the concrete stairs after the entire concrete frame has been erected. Unfortunately, with this system of construction, especially where the stairs are arranged so that one flight depends on another for support, it frequently happens that little attention is paid to the bonding of the stair construction with the concrete work already in place. Bonding rods are forgotten, and often no provision is made for the bearings of the landing slabs; thus, much cutting and patching must be done, and the result is a weakened construction.

Again, the construction may be carried on in such a way that the stairs must be supported by the brick walls of the stairway enclosures. Usually, these walls are run up with the necessary corbelings, and holes for beam bearings are left in them; also, the forms of the concrete stairs are formed within the stair shafts. With stairs properly designed, this kind of construction is very rigid.

45. Sections of Reinforced-Concrete Steps.—Reinforced-concrete steps are formed in several ways, the usual section for such steps being illustrated in Fig. 40. In (*a*) is shown a section of reinforced-concrete steps with a horizontal tread and a vertical riser. This type of step may be easily constructed and finished. Generally, the angle between the riser and the tread is finished with a slight fillet, and the nosing of the step is rounded off as shown in the figure.

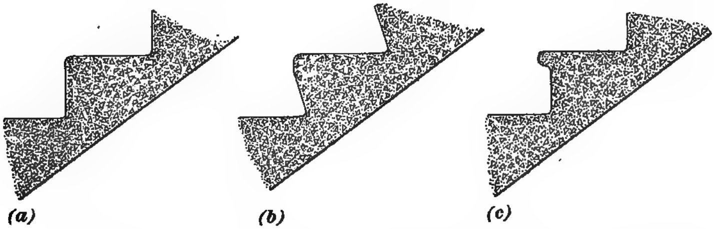


FIG. 40

As reinforced-concrete steps are used mostly in the industrial type of building, though they have been constructed in hotels and fine residences, the space apportioned to them is generally restricted. Therefore, any design of step that will cut down the run, or horizontal extent, of the steps is of advantage. In such instances, the step section shown in (*b*) is sometimes employed. By arranging the step with a horizontal tread and a sloping riser, the tread is widened; or, with the same width of tread, the run of the stairs may be reduced. This section of step is nearly as easily constructed as the one shown in (*a*), though its appearance is not so good.

In (*c*) is shown the section of a concrete step that presents a better appearance than either of the sections described.

This step is formed with a nosing that has the advantage of a wide tread and gives a finish to the stair construction. The nosing, however, is usually worked on with the finish of the step, instead of being formed in the actual concrete construction, and is liable to be knocked off or marred with hard usage.

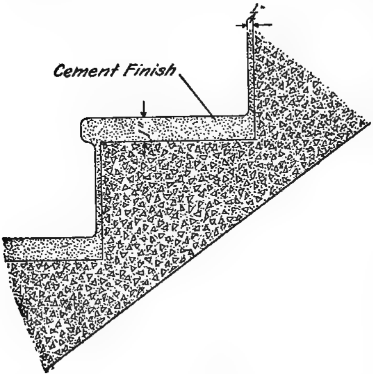


FIG. 41

46. Finish of Concrete Steps.—In finishing concrete steps with cement, the material is placed on the steps in the manner indicated in Fig. 41. Usually, the cement finish, which consists of 1 part of cement and 1 part each of sharp sand and crushed rock, is placed directly upon the concrete work while it is still fresh. Generally, a capping 1 inch in thickness is placed on top of the tread, and the finish reduced to a mere skin coat of about $\frac{1}{8}$ or $\frac{1}{4}$ inch on the face of the risers. The molding and rounding of the nosing is formed in the cement finish.

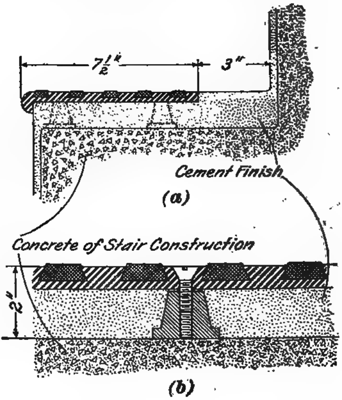


FIG. 42

47. If the steps are on a main stairway of an important building, cement does not make a good finish, because it is dusty and does not present the best appearance. One of the best ways of finishing concrete steps located in such buildings is to use safety treads, as illustrated in Fig. 42 (*a*). These treads are of the section shown, and are made of wrought iron or steel, with lead or carborundum secured in grooves or fastened in some other way. They are usually about 5 to 7½ inches in width and are furnished with a nosing piece, as shown.

In placing these treads upon concrete steps, cast-iron blocks, as indicated in (*b*), are embedded in the concrete work or cement finish, and the treads are fastened to them by countersunk machine screws. These treads form a finish at the nosing, make a good appearance on the steps, prevent the concrete steps from being worn into unsightly holes by the traffic upon them, and also overcome any danger of slipping.

48. Frequently, concrete steps are finished with rubbed slate or marble treads with molded nosings, and if a fine finish is desired, the same material is used for facing the risers. The marble or slate should be set in cement, and is best secured by fastening it with screws to castings or blocks embedded in the concrete work.

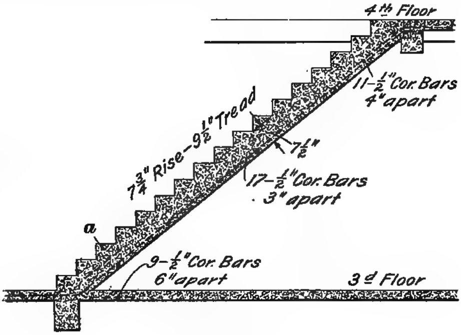


FIG. 43

Wood may be used for finishing reinforced-concrete steps by embedding nailing pieces in the concrete work and finishing the steps with hardwood treads and risers with molded nosings. This finish, however, is hardly consistent with the material of construction, the slate or marble being more suitable.

49. Simple Concrete Stairs.—Reinforced-concrete stairs of simple construction are illustrated in Fig. 43. Here, the stairs extend from floor to floor, and are supported upon header beams, which receive them at the foot and the top. Such stairs are reinforced with rods placed near the soffit, the rods extending the length of the flight and lapping over the beams of the floor systems and stair landings.

In calculating the strength of such stairs, the distance from the internal angle made by the riser and the tread to the soffit is considered as the thickness of a reinforced-concrete slab; also, the reinforcement is of an area and spacing that, with this thickness, will give the necessary resistance to equal the bending moment. Frequently, small rods *a* are placed so as to run parallel with the steps, as shown. These rods act as shrinkage rods and in no way increase the strength of the flight.

50. Stairs to be placed in a brick enclosed shaft, with landings at the top and the bottom of the flight, are constructed as shown in Fig. 44. Brick corbeling is usually provided in the shaft to form a bearing for the landing slabs, and the reinforcing rods extend from the wall bearing of one landing slab to the wall bearing of the other, running throughout the flight. The reinforcing rods are bent to the necessary shape to form this continuous reinforcement. In this construction no header beams are used.

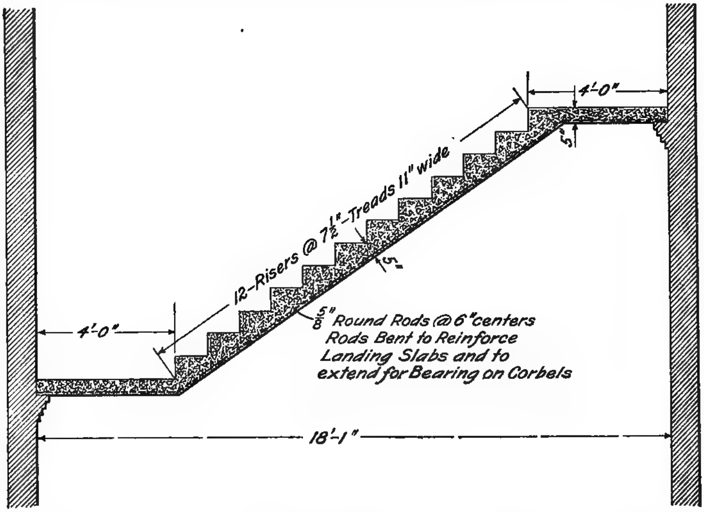

4'-0"
12-Risers @ 7½"-Treads 11" wide
5"
5"
⅝" Round Rods @ 6" centers
Rods Bent to Reinforce
Landing Slabs and to
extend for Bearing on Corbels
4'-0"
18'-1"

Fig. 44

51. If the stairs are to be constructed after the building has been completed, and are to extend from floor to floor and rest on concrete header beams, the beams are made with a rabbet *c*, as shown in Fig. 45. In such cases, the reinforcing rods *a* of the floor system are left to protrude, so that they can tie in with the reinforcing rods *b* of the steps to be constructed later. As will be noticed, the rabbeted beam forms a bearing for the top of the stairs.

52. Stairs With Half-Pace Landing.—A type of stairs much used in all buildings of commercial or industrial type is shown in Fig. 46. Stairs of this kind are necessary where the distance from floor to floor is 12 or 14 feet, and where it is required that they occupy as little room as possible in the building.

As shown in the plan (*a*), the stairway is composed of two half flights, with an intermediate landing between floors. Sometimes the stairs are arranged with an open well that is 8 or 10 inches in width, but more frequently the face of the string of one flight is directly over the face of the string of the flight below. The stairs are usually constructed and reinforced as shown in the section (*b*). By arranging reinforced-concrete beams at *a* and *b* so as to extend across the stair shaft, the construction is greatly strengthened, for then the span of the stairs can be figured as the distance between these two beams, and the span of the slab can be considered as the distance from the beam to the wall. The wall, as at *c*, receives the slab at the first floor level. Where the wall is built afterwards, the slab is received upon a reinforced-concrete beam that is supported at the corners of the shaft either by temporary shoring or by concrete posts or columns.

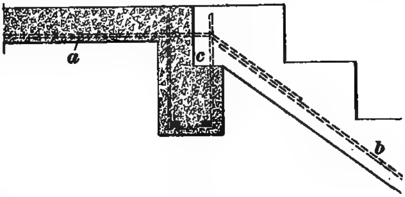


FIG. 45

Sometimes the reinforcing rods are arranged to run through the slab and the stairs, as shown, or else the reinforcement for the stairs extends only over the reinforcement of the beams *a* and *b* and the slab rods are independent, extending from the wall to these beams. Shrinkage rods are placed through the slab, as shown at *d*.

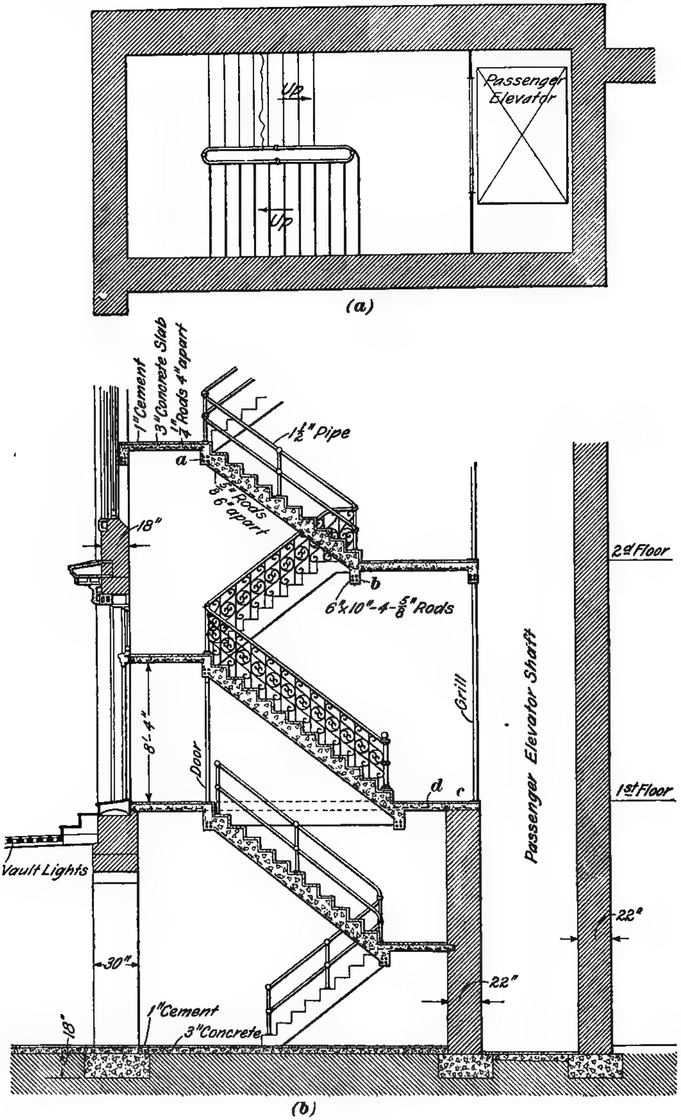

Up
Up
Passenger Elevator
(a)
1"Cement
3"Concrete Slab
1/4" Rods 4" apart
1½" Pipe
a
3/8" Rods 6" apart
18"
b
6"×10"-4-5/8" Rods
2d Floor
Passenger Elevator Shaft
Grill
8'-4"
Door
d
c
1st Floor
Vault Lights
22"
22"
30"
1"Cement
3"Concrete
18"
(b)

FIG. 46

53. Stairs With Center Well Hole.—In many buildings the bays are arranged so as to be nearly square, and as it is convenient to arrange the stairs in a stair well built between piers and columns so as to occupy the space of a bay, the type of stairs with a central well hole, as shown in plan in Fig. 47, is extensively used. Stairs of this construction can also be conveniently used around the elevator shaft, it being customary to arrange the passenger elevator inside of a stair well in this manner.

The method of constructing stairs of this kind is illustrated in Fig. 48, the section being taken along the lines $x\,y\,z$, Fig. 47. In constructing such stairs of reinforced concrete, the several flights between floors act as supports for one another. The flight a, Fig. 48, is considered as spanned between the beams b, and acts as a support for the intermediate flight c. This flight is supported by the landing d, which is either built into the wall or supported upon corbels. Therefore, the upper flight e can be looked upon as suspending, or tending to hold up, the intermediate flight c. Sometimes the stairs are built 4 inches into the wall, and in this way gain some of the benefits of a cantilever beam.

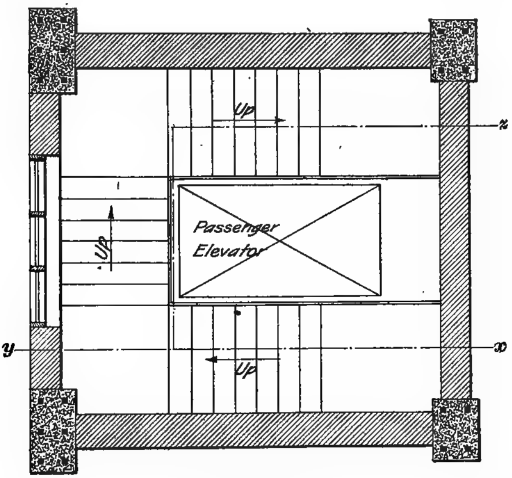


FIG. 47

In designing such stairs, the extreme conditions encountered on any flight should be considered, and the thickness of the stair slab—that is, the distance from the internal angle made by the tread and riser to the soffit of the stairs—as well as the reinforcement, should be proportioned for these extremes. Such calculations will give the basis of design for such stairs, and where necessary to make the construction practical, they may be modified, owing to the numerous safety factors existing on account of the monolithic character of the work, and because of the support given the stairs by the wall and floor constructions.

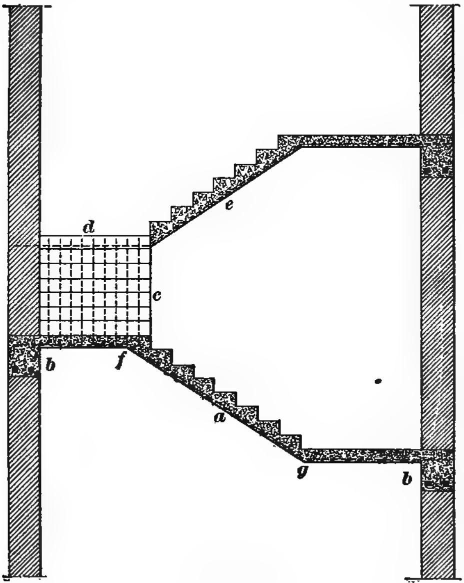


Fig. 48

54. Frequently, the stairs can be greatly strengthened by the use of header beams at such places as *f* and *g*, Fig. 48, where they do not interfere with the headroom nor the elevator shaft; or, a reinforced string can be run along the edge of the stairs. Such a string molded in conjunction with the reinforced-concrete stairs is shown in Fig. 49. A reinforced-concrete string, or curb, provides a means for receiving the elevator grill or stair railing, and likewise prevents dust or dirt from falling down the well. Such a curb should, however, not be run across the landings where the sliding doors to the elevator enclosure are located, as they interfere with the formation of the sill as well as the installation of the sliding doors. When no elevator is present, however, they may be made continuous.

55. Spiral Stairs.—Several types of spiral stairs have been constructed in reinforced concrete. In some instances the spiral has been continuous, in the form of a helix, around a circular well hole, and in others straight flights have been combined with winders, so that a portion of the run is straight and a portion spiral. As a rule, spiral stairs in reinforced concrete are too expensive in cost of form construction for use in commercial buildings, though they may be used if an imposing vestibule or stair entrance is desired.

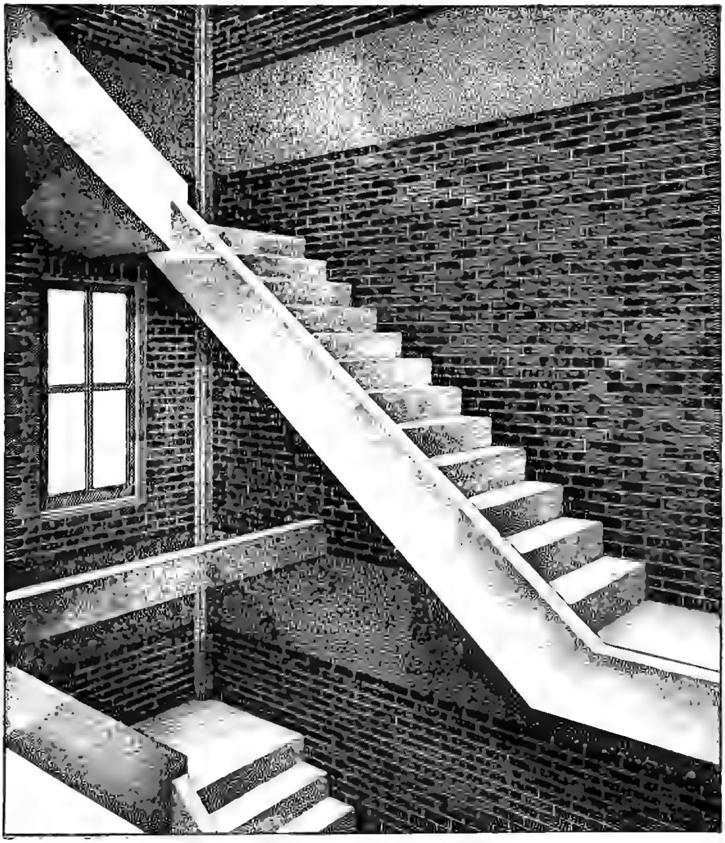
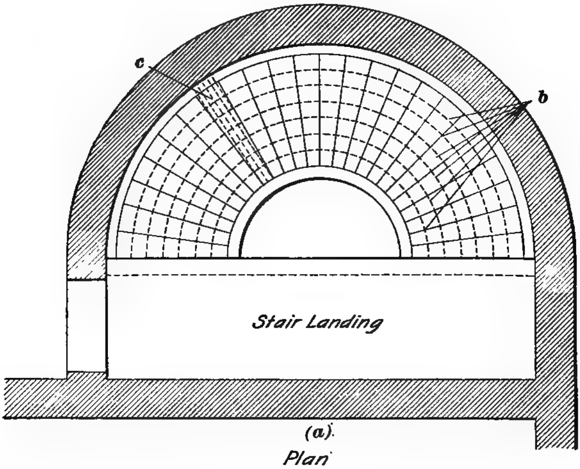
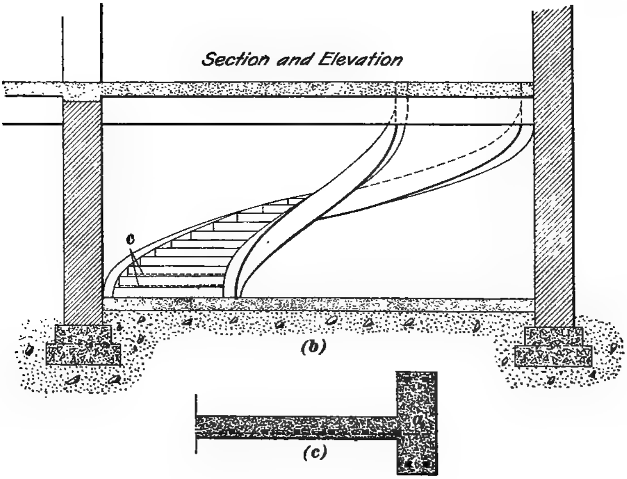


FIG. 49

56. A type of spiral stairs that could be used in such instances as the foregoing is illustrated in Fig. 50. From the plan view (*a*) and the sectional elevation (*b*), it will be seen that these stairs are constructed with a well and an outside string of reinforced concrete. These strings are reinforced at



(a)
Plan



Section through Step Slab and String

FIG. 50

the top and the bottom with steel rods and are usually combined with the slab portions of the stairway, as shown at *a* in the sectional detail view (*c*). Great care must be taken in designing such a stairway, as it is usually a particular feature of a building and is not one of a number of elements in the construction. Besides reinforcing the concrete strings, it is best to run reinforcing rods *b* in the form of a helix along the stair slab and parallel with the string. The stairs should likewise be reinforced transversely with rods *c*, in order to avoid shrinkage, and also give the stairs and stair slab transverse resistance between the strings. If the strings can be made of a good size and are well reinforced, the stairs may be made of the section illustrated in Fig. 51, in order to minimize the dead-weight and lessen the transverse stress upon the strings.

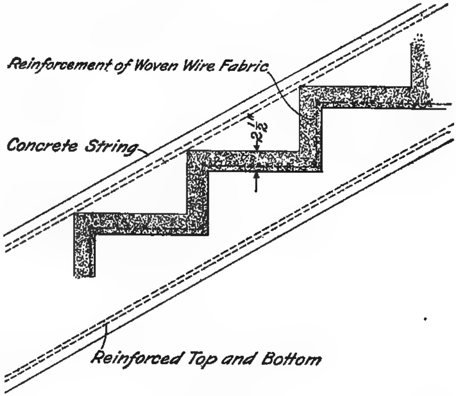
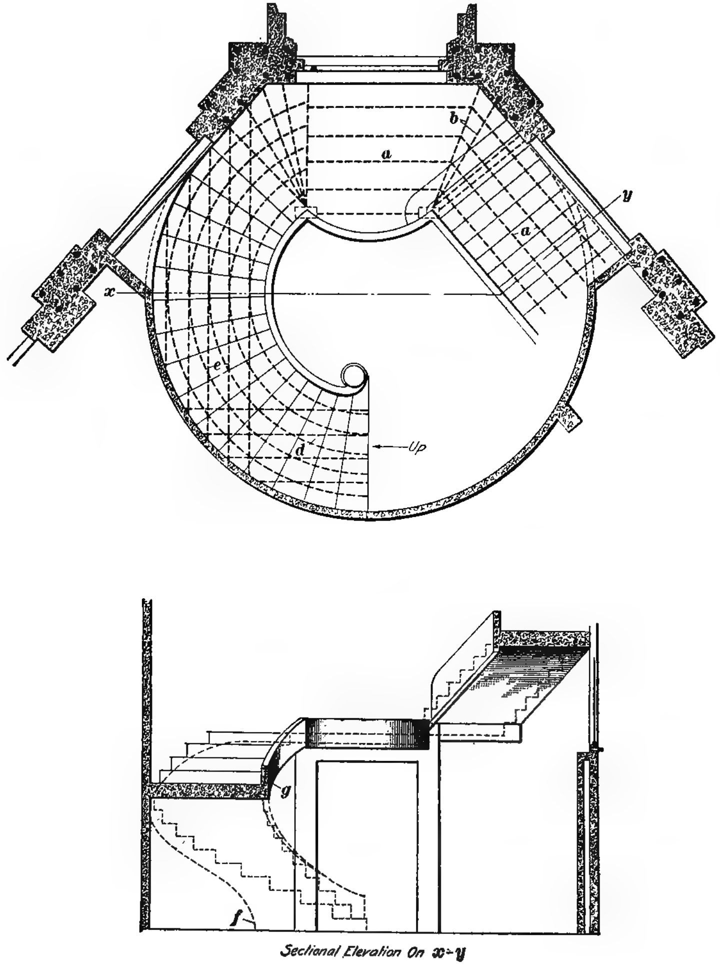


FIG. 51

It is hardly possible to make theoretical calculations cover the design of a stair of this character. The stresses can only be approximated, and the necessary factor of safety will have to be obtained by careful designing and the correct placing of the steel reinforcement, it being best to have this well distributed and extending in all directions.

57. In the type of stair shown in Fig. 52, where a straight flight connects with a spiral portion, the rods *a* of the straight flight can run through and rest upon cross-rods *b*. This forms a heavily reinforced portion of the slab, from which the spiral portion of the stairs can start. Other rods *d* and *e* can be used to advantage in the construction of such a stair. The stairway is greatly stiffened by the use of a small spandrel *f* at the commencement of the flight. By reinforcing the run of the stair with an outside closed string *g* of reinforced concrete, the stairs are improved in appearance and greatly strengthened. The string also acts as a structural member from which to start the balustrade, which may be



Sectional Elevation On x-y

FIG. 52

of ornamental ironwork, with a bronze pipe or hardwood molded rail.

Concrete spiral stairs with architectural pretensions may be finished with ornamental plaster work or covered with scagliola. The ornamental moldings, band courses, and other ornaments need not be attempted in the rough concrete work, but can be a portion of the finish.

58. Concrete Stairs With Separately Molded Members.—It is possible, though not customary, to construct reinforced-concrete stairs with separately molded members, as illustrated in Fig. 53. In this figure, the string at (*a*) is either molded and afterwards set up or is molded in place, being properly reinforced at the top and bottom and cast with a ledge *b*, shown in (*c*), to receive the steps. The reinforcing rods at the top and bottom should be tied together with wire stirrups, so as to prevent the string from cracking by shrinkage or in handling. The steps are molded of the form shown in (*b*), and may be arranged with a nosing as at *a*. When these separately molded steps have become properly set and seasoned, they are placed upon the string

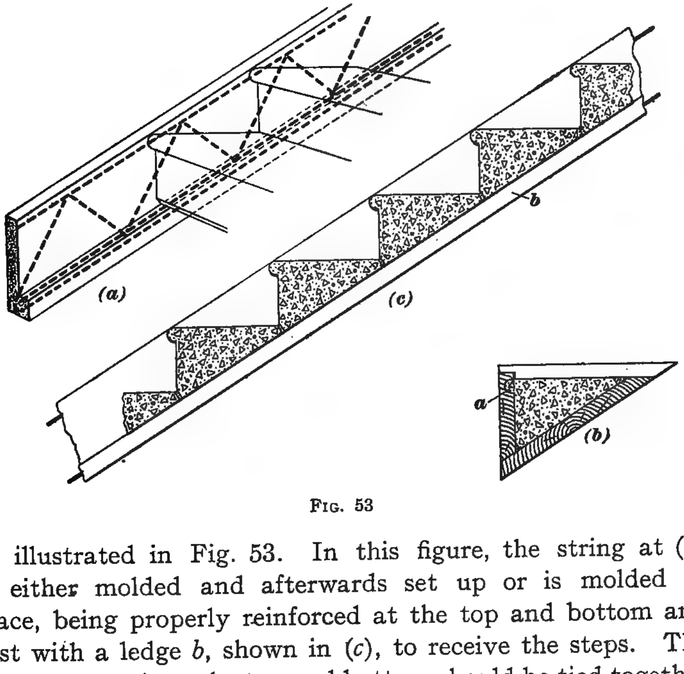


FIG. 53

and bedded in cement mortar, as shown in (*c*). As the steps span from string to string, they can be secured against breakage or cracking by extending light rods throughout their length. When erected, the steps can be finished with a cement coat, or, if care is taken in their molding, they will be sufficiently smooth for ordinary purposes without any extra finish.

59. Construction of Hand Rail for Concrete Steps. The hand rail generally used on reinforced-concrete steps consists of galvanized-iron pipe that is put together with galvanized-iron screw fittings. The usual construction of such a hand rail is illustrated in Fig. 54. In this figure, the hand rail *a*, the guard-rail *b*, and the stanchions *c* are made of 1½-inch galvanized pipe. With inferior construction, the hand and guard-rails are run through the fittings and are not fitted into them. This type of construction is not so firm or rigid as that shown in the figure.

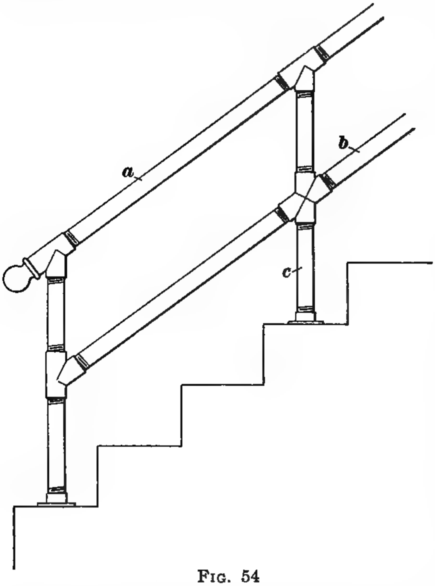


FIG. 54

Many methods are employed to secure the stanchions in the steps. Sometimes a pipe sleeve of larger size is embedded in the concrete steps, as shown in Fig. 55 (*a*), and the stanchion is leaded into the socket, being finished with a flange, as shown. Frequently, a pipe socket is threaded at the upper end, and the stanchion is secured to it with a coupling and finished with a loose flange, as shown in (*b*). In each of these methods, the stanchions are placed on the top of the tread. If, however, it is desirable to gain the full width of the stairs, the stanchion is supported as shown in (*c*). Here, a threaded pipe socket *a* is placed through the form board used in molding the side, or cheek, of the steps; then, when the form board is removed, the threaded end of the socket projects and allows the stanchion to be supported by a threaded **L** fitting screwed on the socket.

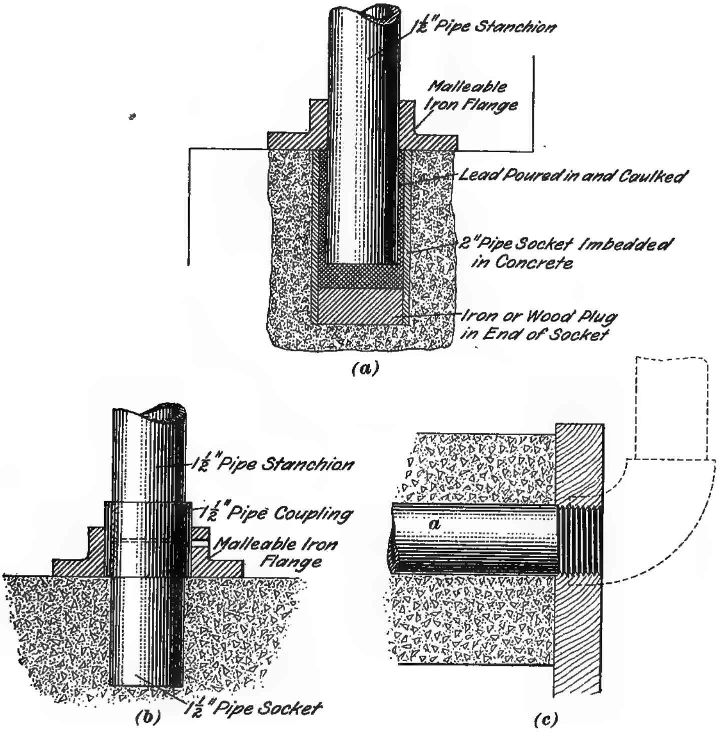


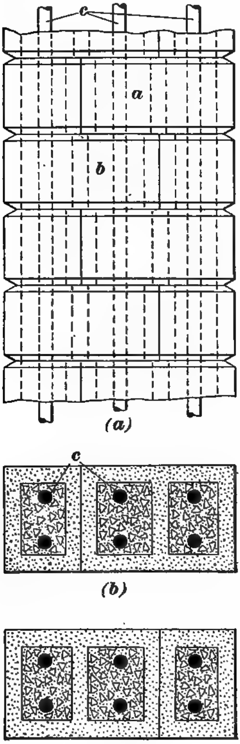
FIG. 55

Ornamental iron rails may be used on reinforced-concrete steps of a more pretentious character than the preceding. In such designs, the uprights of the rail are secured by leading them into pipe sockets previously placed in the concrete work, or by drilling the concrete and leading or otherwise securing the uprights.

REINFORCED-CONCRETE BLOCK CONSTRUCTION

60. The construction of reinforced-concrete exterior wall piers and spandrels has been attended with several difficulties, namely, the obtaining of a good architectural finish, the cost of the wood forms, and the difficulty of constructing this work in freezing weather without having the surface of the piers materially damaged by frost. In order to avoid these difficulties, a type of building constructed with piers built of hollow concrete blocks, the holes of which are filled with concrete, and reinforced with steel rods, has been designed, and, in several instances, erected.

61. In Fig. 56 (*a*) is shown a typical pier of this construction, the two courses *a* and *b* of the piers being illustrated in (*b*) and (*c*). As will be observed, each course consists of two hollow concrete blocks, with the holes in the blocks in each course coming over those in the course beneath. In this way, reinforcing rods *c* can be placed and the holes in the blocks filled with concrete. By this construction is provided a composite pier consisting of the concrete blocks molded previously and laid in cement mortar, in a manner consistent with the best practice, and reinforced-concrete cores, which add to the bearing strength of the piers as well as to their lateral stability.



(*a*)

(*b*)

(*c*)

FIG. 56

In designing such piers, the standard size of blocks made by the particular cement block machine to be used must be adhered to, and the piers must be of a width and a thickness to correspond with the dimensions of the blocks. Many block machines are limited as to the height of block that they will hold, so that the story heights must be worked out to suit the height of the blocks when laid. This feature is important to observe, as the height of the blocks fixes the location of the beam, girder, and lintel bearings. This may be explained by means of Fig. 57, which shows the lintel and floor construction in conjunction with a cement-block pier. The lintel supports a spandrel and is of such height as to permit the floor slab to come at the horizontal joint of the block course, as shown at *a*, and the girder that extends into the wall is arranged with regard to depth so that it will bear upon the next course but one below the lintel. It can thus be seen that the floor height is regulated by the height of the blocks.

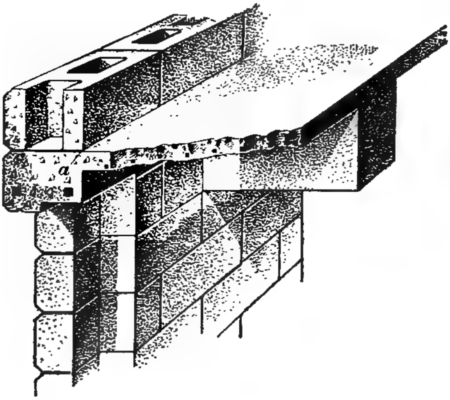


Fig. 57

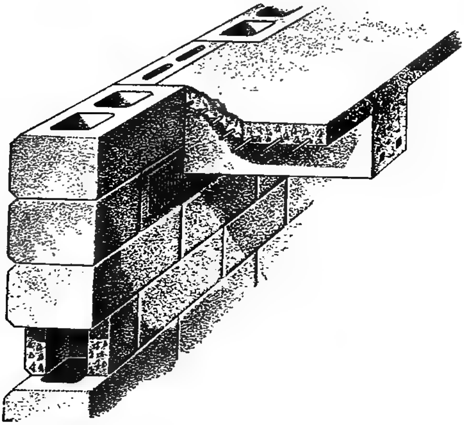


Fig. 58

62. If it is not possible to arrange the depth of the beams so that they will work out with the horizontal joint of the block, the end of the beam can be constructed in the manner shown in Fig. 58. By arranging the forms so that the space below the beams will be formed of concrete, the bearing of the beam is carried down to the top bed of the block below.

The spandrels in this construction may be made either of solid concrete, as illustrated in Fig. 59 (*a*), or of hollow concrete blocks constructed as shown in (*b*). It is customary to provide on the top of the spandrel a solid artificial stone sill made in short sections in an ordinary cement-block machine.

63. Piers constructed in the manner just described will not have the same bearing strength as a monolithic pier of reinforced concrete. It is hardly good practice to calculate that piers of this character will carry safely more than 300 pounds to the square inch, for while the reinforced-concrete cores, if solidly built, would be capable of resisting safely 500 pounds to the square inch, it is more than possible that this filling would not be carefully done.

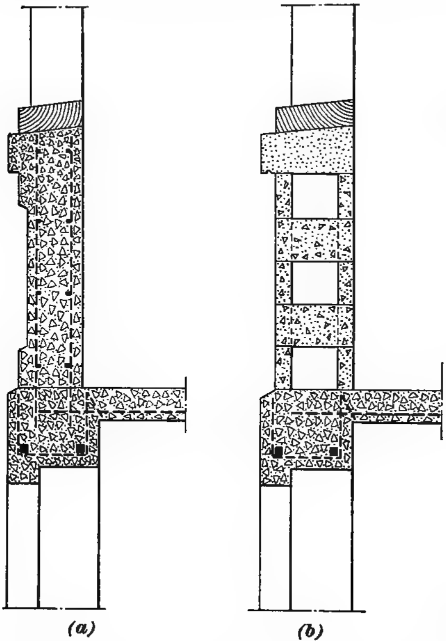

(*a*) (*b*)

FIG. 59

Piers of this character can be constructed in two ways. One method consists in first building up the piers. Then, after the piers have reached their height, reinforcing rods are inserted and the concrete filling of the cores is tamped in the best manner possible under the conditions. It is necessary to use almost a grout for this purpose. The other method consists in placing the rods and threading the blocks over their top, filling in each course as the work progresses. This kind of construction is more troublesome than the one first mentioned, but is likely to result in better work.

64. In constructing buildings with blocks in the manner just explained, it is customary to use blocks that look like dressed stonework, and to cover the joints by molding the blocks with either rabbeted or chamfered edges, as shown in Fig. 60. In forming the blocks with chamfered edges, as in (*a*), a slight bevel of about 1 inch on the side is generally used, so as to give good joint-line effects to the piers, and not give the effect of rustication. Many machines are arranged so that the mold has to be filled from the top, in which case the metal edge that is placed in the form to give the chamfered effect is bothersome, as it interferes with the tamping of the cement. Where blocks with a rabbeted edge are used, the blocks are made as indicated in (*b*), the rabbet being provided on the lower edge of each block, so that each block is alike and may be laid up as indicated.

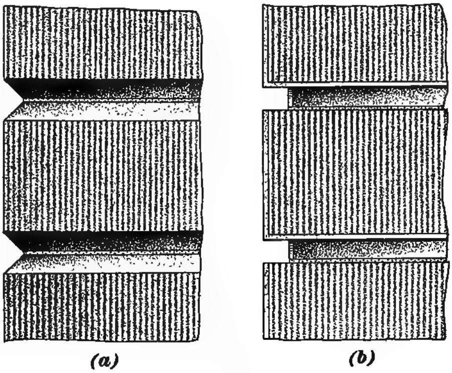

(*a*) (*b*)

Fig. 60

REINFORCED-CONCRETE BUILDINGS

(PART 2)

CORES FOR REINFORCED-CONCRETE COLUMNS

USE OF COLUMN CORES

1. In high concrete buildings with large floor spans and heavy loads, the columns of the lower stories, if of ordinary reinforced-concrete construction, must be of large size. In order to reduce the size of the columns where such conditions exist, it is common practice to embed in them **column cores** made of structural-steel shapes. Although such columns are similar to steel columns fireproofed with concrete, their strength is not ordinarily figured by steel-column formulas, but is obtained by using the safe allowable unit stress upon the steel in direct compression, no consideration being paid to bending stresses due to flexure in the column.

In calculating the strength of the steel cores in this manner, it is considered that the surrounding concrete, together with any rod reinforcement and ties, provides sufficient lateral resistance to prevent the deflection of the steel core. Consequently, in the design of such cores, the radii of gyration of the section is unimportant, it being necessary to obtain only a sectional area that, when multiplied by the allowable unit stress of the steel in direct compression, will equal the column load.

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Sometimes cast-iron columns are embedded in concrete in the same manner as the steel cores. The best practice, however, in using cast-iron column cores is to figure their strength by the column formula, as, on account of the unreliable nature of the material and the likelihood of their being badly twisted or out of plumb, it is not safe to base the calculations on direct stress.

TYPES OF COLUMN CORES

STEEL CORES

2. Although a steel column core for reinforced-concrete construction may be of any shape and may resemble any of the well-known types of steel columns, such as the plate and angle, the **Z** bar, or the channel section, yet a section should ordinarily be adopted that has the least dimension from out to out of edges. The section should be such, however, that the concrete can readily be tamped around it to fill the form.

3. Built-Up Core.—An excellent type of steel-core section for reinforcing concrete columns is shown in Fig. 1 (*a*).

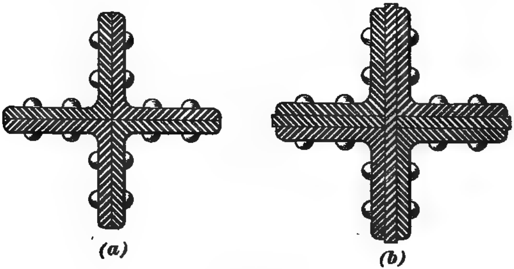

(*a*) (*b*)

Fig. 1

This section is built up of four angles placed back to back, but sometimes rolled plates are placed between the angles, as shown in (*b*). By the use of heavy angles and plates, a section of this kind can be built up to contain as much as 50 square inches, and the outside dimensions of the concrete column with such a core need not be more than 18 or 20 inches.

In Fig. 2 is shown the usual additional reinforcement required in the construction of such a column. This extra reinforcement consists of round or square bars, usually four or eight in number, that range from ¾ inch to 1⅓ inches in diameter. These bars give additional lateral strength to the reinforced concrete, and when properly arranged, with good laps at each floor, they provide an additional tie where the continuity of the column is broken by means of splice plates. The rods are tied together at intervals with ¼-inch round iron or wire ties, which, for ordinary loads, should not be placed farther apart than 24 inches from center to center.

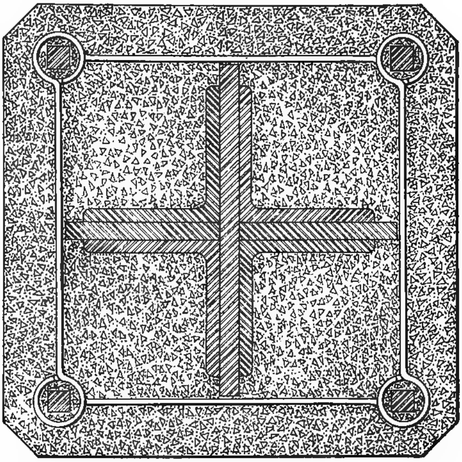


FIG. 2

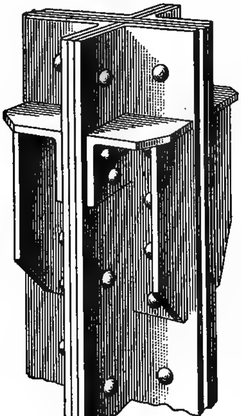


FIG. 3

4. In designing structural-steel column cores for reinforced-concrete construction, it is customary to provide brackets formed of structural shapes. These brackets are riveted or bolted to the columns in such a position as to provide bearings for the reinforced-concrete beams and girders. Brackets of this kind are shown in Fig. 3. They should be located in such a position below the floor as to receive the ends of the lower reinforcing rods for the beams or girders, and to act as a bearing for these rods when they are placed preparatory to putting the concrete in the forms. Where a column is to receive both beams and girders, it is best to put the brackets at the level of the lower rods of the girder reinforcement and to allow the concrete to fill the space in the column form between the top of the bracket and the lower rods of the beam reinforcement so as to make a bearing for these rods.

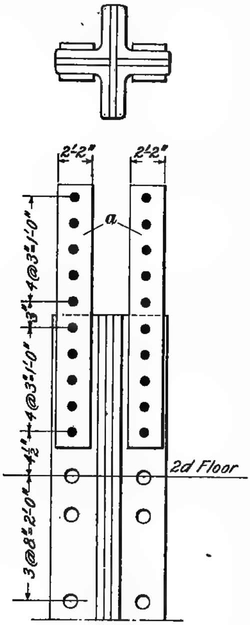


Fig. 4

The brackets are best made of angle clips and plates constructed as indicated in the figure. Enough rivets should be used to overcome the reactions from the ends of the beams or girders. Care must be observed in the construction of such brackets to see that they do not interfere with the rod reinforcement of the column. It is necessary sometimes to cut off the corners of the horizontal flange of the brackets in order to allow the reinforcing rods of the column to extend beyond them

5. In the design of the steel cores, it is good practice to run them in one length through two stories and to make the splices always above the level of a floor, so that the joints will extend above the concrete work of the lower story.

A typical splice and its position with reference to the concrete work at the floor level are illustrated in Fig. 4. The junction is usually made secure by means of the splice plates *a*. The splice may be made with either bolts or rivets, and the ends of the abutting column cores should always be milled true and at right angles to the axis of the core.

6. The end of the uppermost column core is sometimes provided with a cap plate *b*, which is held to it by means of angle clips, as shown in Fig. 5. Such a plate is considered necessary by some engineers in order to provide a bearing for the first reinforced-concrete column without a core. As this reinforced-concrete column, however, is assumed to transmit only 500 pounds to the square inch of cross-section, and as the concrete at the top of the uppermost steel-core column is rigidly reinforced in all directions by the beam and girder junction at the column, there is hardly any necessity for providing this cap plate.

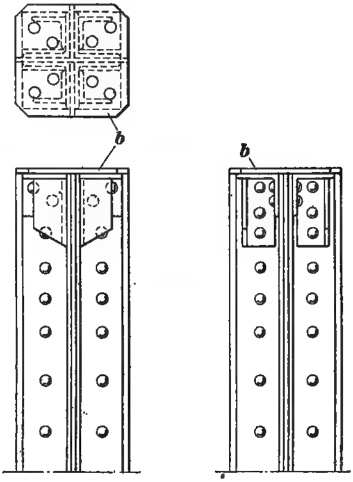


Fig. 5

7. The construction of the steel column core at the base should be thoroughly studied. In order to realize their strength, such cores require an extensive footing area,

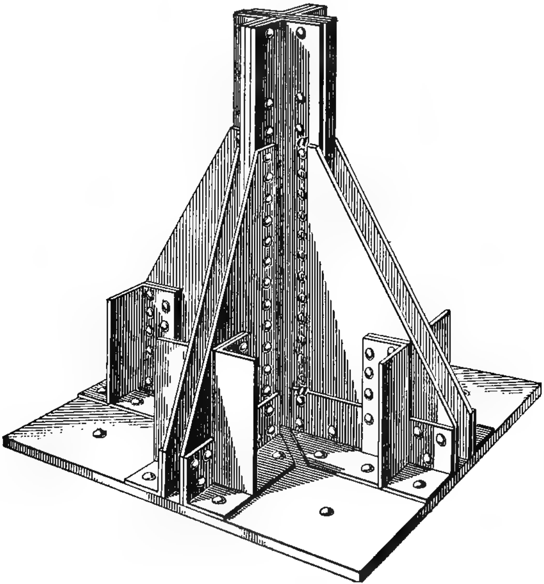


Fig. 6

which must be either a reinforced-concrete or steel-beam grillage; and in order to transmit the stress in the column to this footing, the base must be spread in the manner shown in Fig. 6. The gusset plates of the base should be amply heavy to transmit the stress, and there should be enough rivets that hold the plate to the column core, as along the line *a a*, to equal in resistance the entire stress upon the gusset plate, which, in this example, is one-fourth the total load on the column.

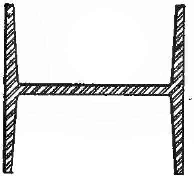


FIG. 7

8. Column Cores of H Section.—Although the built-up core just described is extensively used, yet the rolled section shown in Fig. 7 is particularly adaptable for use as a steel core in reinforced-concrete columns. This type of core, known as the **H section,** resembles an **I** beam that has very wide flanges. **H** sections are made in sizes ranging from 4 to 84.50 square inches, the minimum and maximum sections being shown in Fig. 8. The pound price of such sections is less than that of the built-up cores. By using the **H** section no fabrication, or building up, outside of the brackets that may be required for the bearings of the beams and girders is necessary.

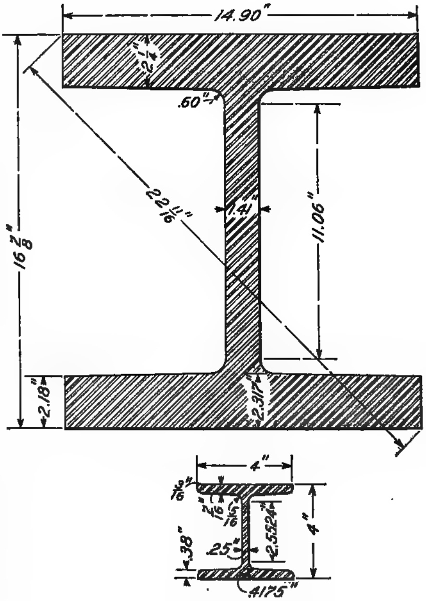


FIG. 8

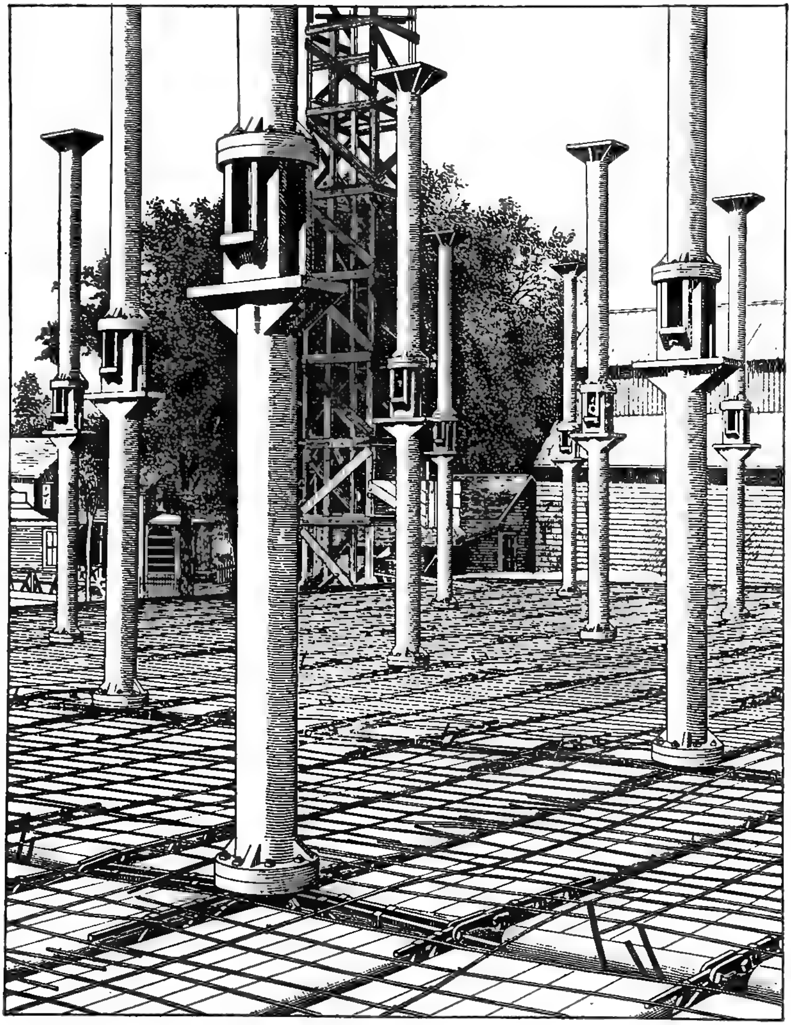


FIG. 9

CAST-IRON CORES

9. Cast-iron columns embedded in concrete and reinforced with longitudinal rods have been used to a considerable extent. The only advantage of using such columns in conjunction with reinforced concrete is that they may be more readily obtained in some localities than structural-steel columns; at least, they are likely to be delivered with less delay than structural-steel work.

A typical cast-iron column used in conjunction with reinforced concrete is illustrated in Fig. 9, where the forms are shown placed, ready for the concrete. As will be observed, the columns are cast with flanges at each story for bolting the several tiers together. At the upper end of the columns are provided girder and beam bearings, and these bearings are so arranged that the concrete enters pockets formed in the column. The cast-iron columns illustrated may be either encased with concrete or left bare. In this case they were not encased.

ARRANGING REINFORCEMENT AROUND CORES

10. Sometimes reinforcing rods are placed in the concrete surrounding the steel column cores. These rods serve to reinforce the concrete additionally and, with the assistance of wire ties, to hold the concrete in position around the cores.

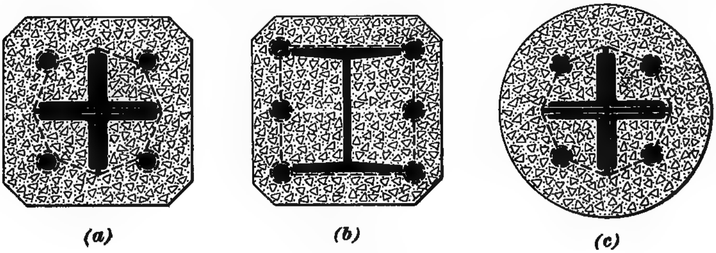

(a) (b) (c)

FIG. 10

In Fig. 10 are shown several methods of arranging the rod reinforcement in columns provided with steel cores. In (*a*) is shown a square column reinforced with a steel column core of star section. The reinforcing rods are placed at the corners of the sections and are tied together with ¼-inch looped wire ties. These wire ties are usually placed not farther apart than the width of the narrowest side of the column, and never at a greater distance than 24 inches. In (*b*) is shown a reinforced-concrete column with a steel core of **H** section. Here, six reinforcing rods, placed as shown, are tied together with looped ties. A cylindrical column is shown in (*c*). The core in this section is star-shaped, and four rods are conveniently arranged between the arms of the core. The ties that hold these rods in position are bent around the edges of the steel column core. By placing the rods between the arms of the core in this manner, the diameter of the column can be reduced.

In all instances there should be at least 2 inches of concrete outside of the reinforcing rods. If steel column cores are used, the reinforcing rods need not be larger than from ¾ to 1 inch in diameter, for while they materially strengthen the column against lateral flexure, their size is not fixed by calculation, and they serve more as shrinkage rods and, together with the ties, as a binder for the concrete than as additional reinforcement.

TYPES OF REINFORCED-CONCRETE BUILDING CONSTRUCTION

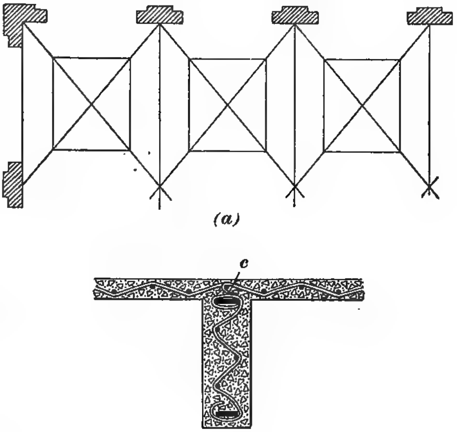
STRUCTURES OF SEPARATELY MOLDED MEMBERS

11. Advantages and Disadvantages of Separately Molded Members.—Systems that employ separately molded members are better known in Europe than in America, and although many advantages are claimed for them, it is doubtful whether they will ever be used to a great extent, because structures erected in this manner do not possess the rigidity of a monolith. The advantages claimed by the advocates of this type of reinforced-concrete construction are as follows:

1. The cost of elaborate form and false work is saved.
2. By building the members in advance of the operation and allowing the necessary time for them to set finally, the erection may be carried on without interruption.
3. Buildings of this type may be erected in winter without danger of frozen concrete, provided the members are previously molded or molded in a shed that is heated.
4. The molded members may be tested, and, if found deficient, they may be discarded.
5. The forms may be used over and over again with slight alterations, thus saving timber and labor.

The disadvantages, however, of this type of construction are that the structure is not so rigid and is more liable to be affected during erection by lateral movement from wind or vibrations than those more monolithic in nature; also, the cost of erecting the heavy members tends to eliminate the saving in the cost of the forms, or false work, and, besides, difficulties are encountered in tying the several parts of structures together at their junctions in a practical manner.

12. Systems Employing Separately Molded Members.—The several systems of reinforced-concrete construction employing separately molded members that are well known in Europe are the *Cottancin*, the *Contaga*, the *Bonni*, and the *Visintini system*. In the first three systems mentioned, reinforced-concrete joists, or beams, are molded and then placed in position, with their ends resting on the walls or columns. Frequently, diagonal members are used, and the connection between the several members is formed by pouring concrete around the junction. In most of these systems, the object is to provide supports of reinforced concrete sufficiently strong to sustain the weight of the floor slab. The web of the beams or girders is allowed to project so that a mechanical bond may be made between them and the floor slab.



(a)

(b)

FIG. 11

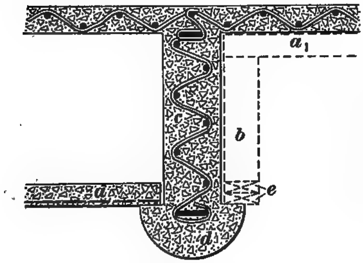


FIG. 12

13. The **Cottancin system** is typical of systems that use separately molded supporting members, and is shown diagrammatically in Fig. 11. In the plan view (*a*) is indicated by single lines the arrangement of the ribs that form the beams or girders, and in the section at (*b*) is shown the beam reinforcement projecting into the slab at *c* so as to bond with the slab construction and form a **T** section.

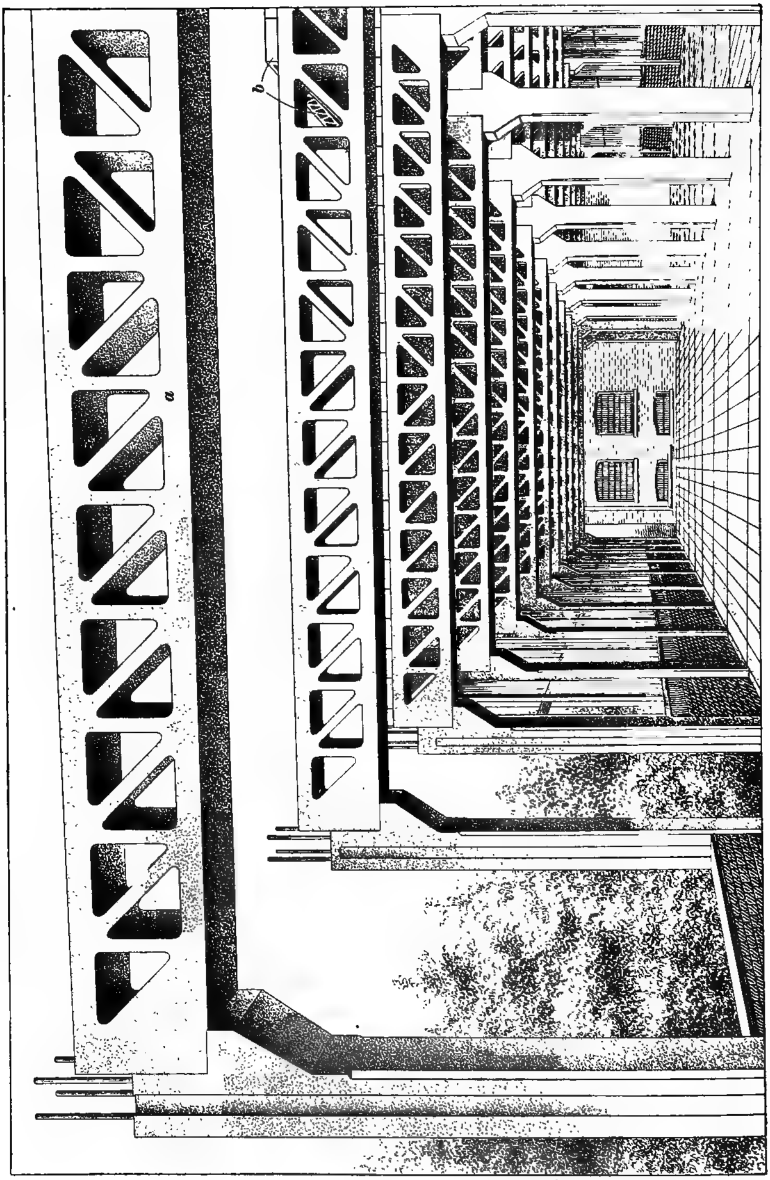


FIG. 13

Where it is proposed to use a ceiling with this type of construction, the method shown in Fig. 12 is employed. As here shown, the beams, or rib sections, *c* are arranged with a ledge *d* that receives a separately molded reinforced-concrete slab *a*. In order to form the floor slab, which is molded in place, the ceiling slab is raised into the position a_1 by means of struts *b* and wedges *e* resting upon the projections *d*. While in this position, the slab forms the centering for the floor construction. When the floor slab is properly set, the ceiling slab is dropped until it rests upon the ribs and occupies the position *a*, where it forms a panel to the ceiling. The ceiling slabs while serving the purpose of centering are prevented from sticking to the concrete of the floor-slab construction by coating them with some material, such as crude oil, to break the adhesion.

14. Among the separately molded and erected systems of reinforced-concrete construction, a well-known one in America is the **Visintini system.** This type of construction is illustrated in Fig. 13, which shows a building about half completed.

In this system, the beams and girders are molded separately and then erected in place, the columns being arranged with brackets to receive the beams and girders. Both the beams and the girders consist of reinforced-concrete truss construction with accurately defined top and bottom chords and oblique web members. The girders *a* are necessarily of large section and are heavily reinforced, while the beams *b* are of smaller section. The beams are placed in contact side by side, so as to form the floor slab spanning over the top of the girder, on which they rest, cement being poured between the abutting ends.

15. The Visintini system of construction was used in erecting a factory building at Reading, Pennsylvania. The structure in question has a central row of columns and is 50 feet 3 inches wide and 200 feet long. The girders have a span of about 24 feet and are placed about 12 feet 6 inches

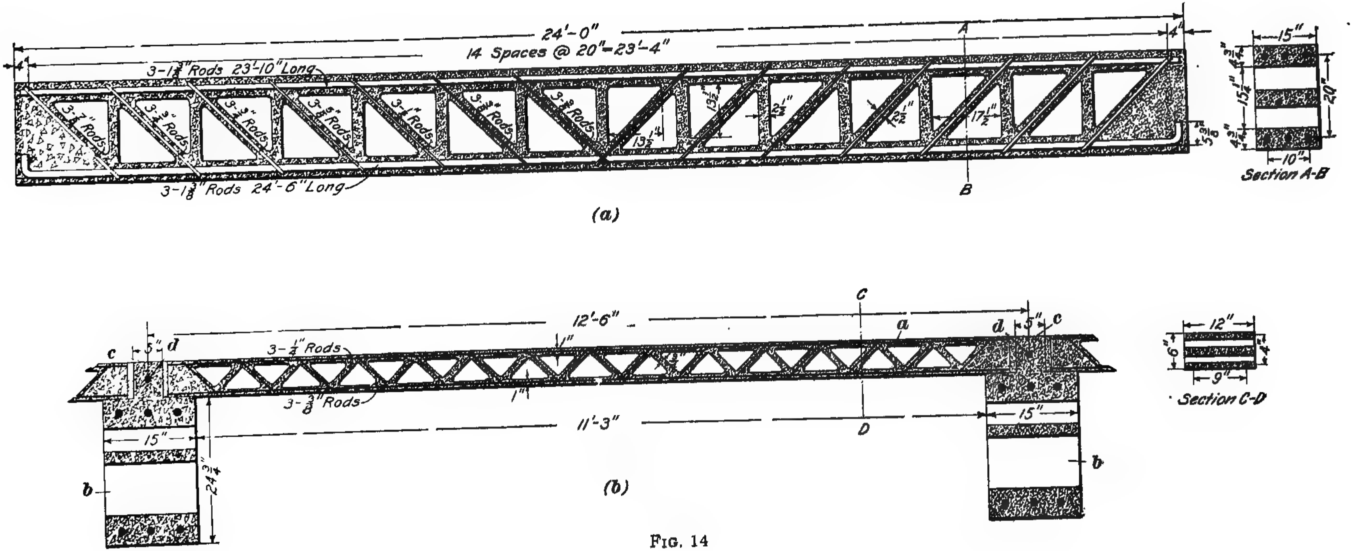
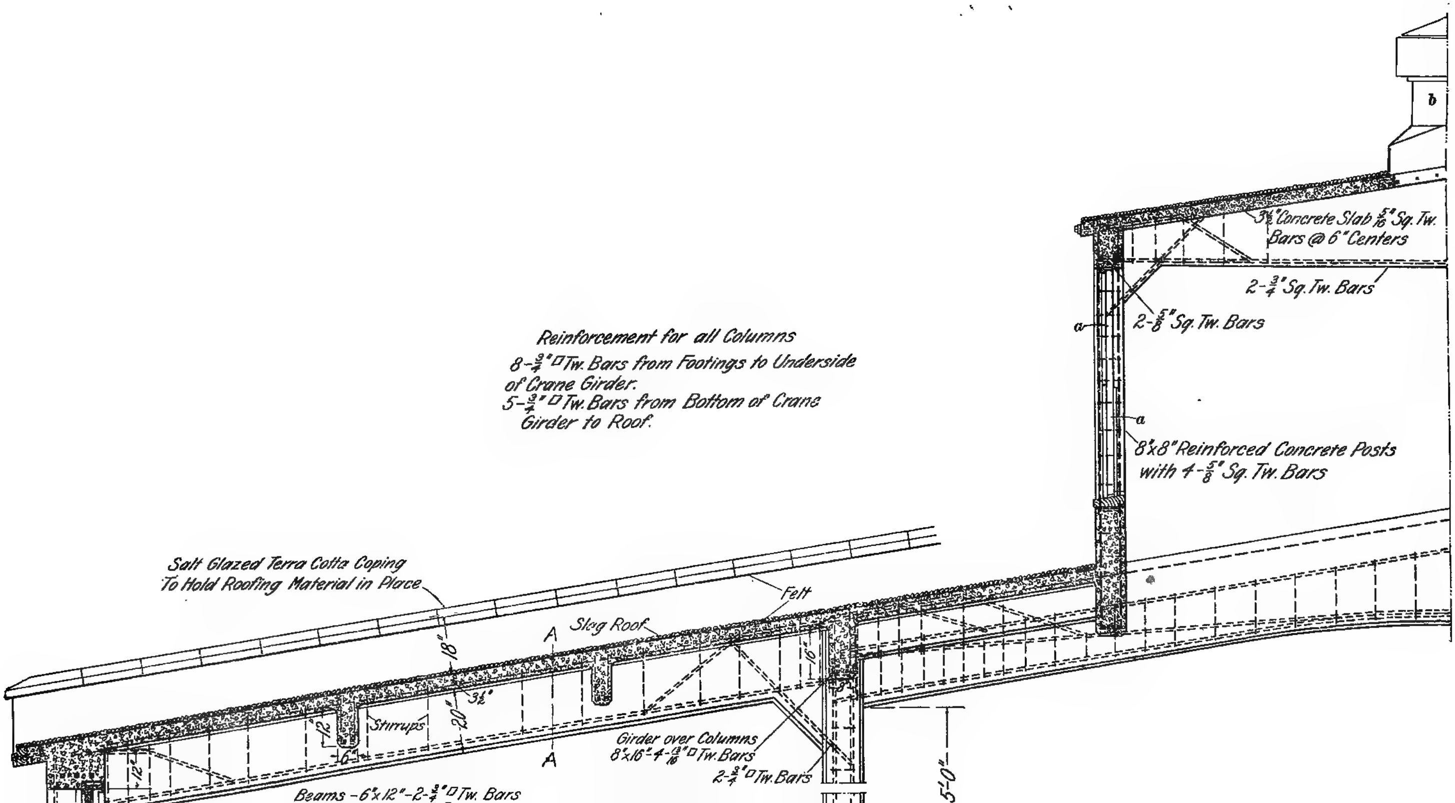

24'-0"
14 Spaces @ 20"=23'-4"
4"
4"
3-1 3/8" Rods 23'-10" Long
3-1 3/8" Rods 24'-6" Long
3-7/8" Rods
3-3/4" Rods
3-3/4" Rods
3-5/8" Rods
3-1/2" Rods
3-3/4" Rods
3-7/8" Rods
13 1/2"
13 1/2"
23 1/2"
22 1/2"
17 1/2"
A
B
5 3/8"
15"
20"
4 3/4"
15 1/4"
4 3/4"
10"
Section A-B
(a)
C
D
12'-6"
11'-3"
3-1/4" Rods
3-3/8" Rods
1"
1"
a
b
b
c
c
d
d
5"
5"
15"
15"
24 3/4"
12"
6"
4"
9"
Section C-D
(b)

FIG. 14



Reinforcement for all Columns
8-$\frac{3}{4}$" □ Tw. Bars from Footings to Underside of Crane Girder.
5-$\frac{3}{4}$" □ Tw. Bars from Bottom of Crane Girder to Roof.

b
3½" Concrete Slab $\frac{5}{16}$" Sq. Tw. Bars @ 6" Centers
2-$\frac{3}{4}$" Sq. Tw. Bars
2-$\frac{5}{8}$" Sq. Tw. Bars
a
a
8"x8" Reinforced Concrete Posts with 4-$\frac{5}{8}$" Sq. Tw. Bars
Salt Glazed Terra Cotta Coping To Hold Roofing Material in Place
Felt
Slag Roof
A
A
18"
16"
3½"
20"
12"
6"
12"
Stirrups
Girder over Columns 8"x16"-4-$\frac{13}{16}$" □ Tw. Bars
2-$\frac{3}{4}$" □ Tw. Bars
Beams -6"x12"-2-$\frac{3}{4}$" □ Tw. Bars
5'0"

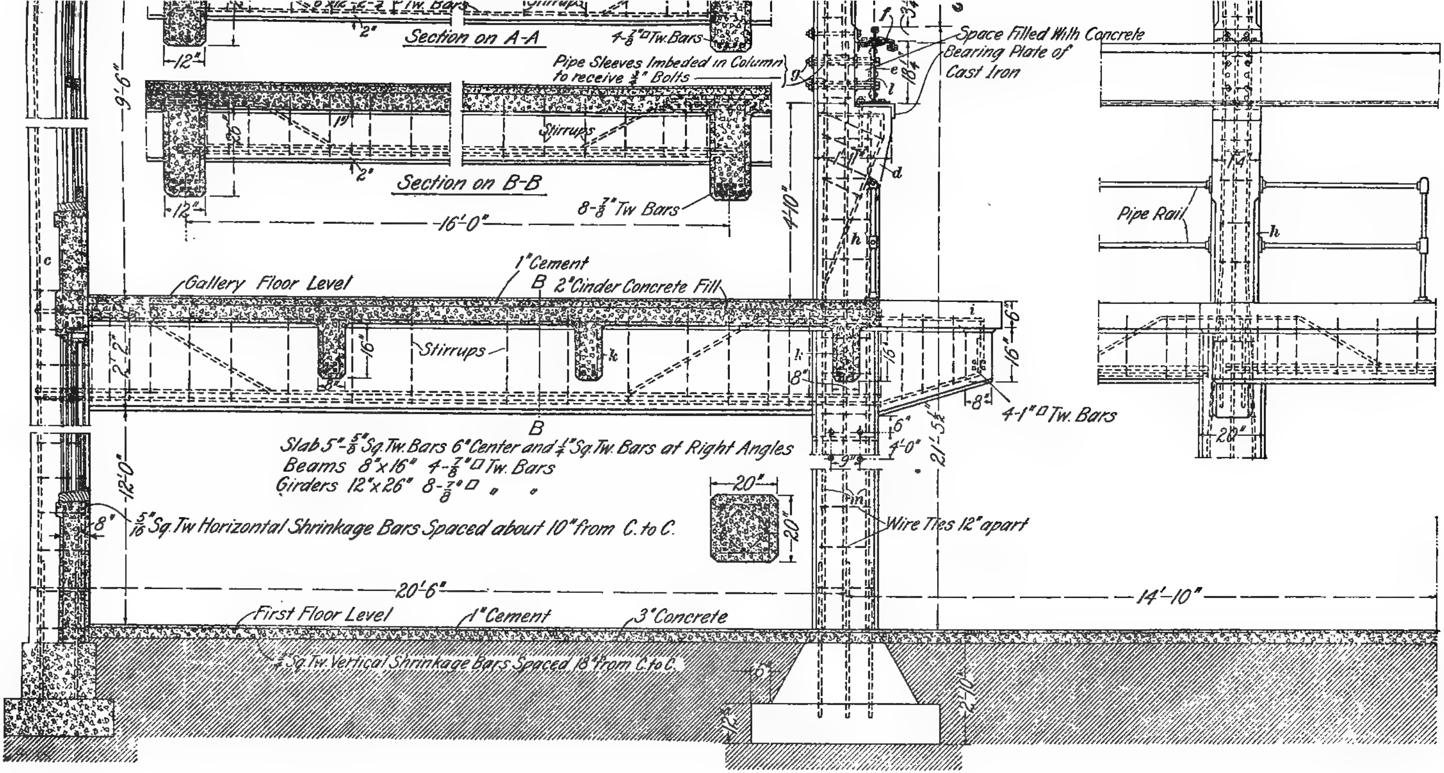

Section on A-A
4-7/8"□Tw.Bars
Pipe Sleeves Imbeded in Column to receive 3/4" Bolts
Space Filled With Concrete
Bearing Plate of Cast Iron
Section on B-B
Stirrups
8-7/8" Tw Bars
9'-6"
12"
2"
16'-0"
4'-10"
18 1/4
Pipe Rail
14"
h
d
e
f
g
l
1" Cement
Gallery Floor Level
B 2" Cinder Concrete Fill
i
6"
16"
Stirrups
k
8"
2'-2"
c
4-1"□Tw. Bars
20"
21'-5 1/4
4'-0"
9"
B
Slab 5" - 5/8" Sq.Tw.Bars 6" Center and 1/4" Sq.Tw.Bars at Right Angles
Beams 8"×16" 4-7/8"□Tw.Bars
Girders 12"×26" 8-7/8"□ " "
12'-0"
8"
5/16" Sq.Tw Horizontal Shrinkage Bars Spaced about 10" from C. to C.
Wire Ties 12" apart
m
20'-6"
14'-10"
First Floor Level
1" Cement
3" Concrete
1/4" Sq.Tw.Vertical Shrinkage Bars Spaced 18" from C.to C.
6"
12"
2'10"

Fig. 15

from center to center, thus making a clear span for the beams of over 11 feet.

The details of the construction of the beams and girders are shown in Fig. 14. In (*a*) is shown the Visintini girder reinforced in the top and bottom chords with three 1⅜-inch rods. The diagonal web members, which are under tension, are reinforced with ⅜-, ½-, ⅝-, ¾-, and ⅞-inch rods, the rods increasing in size toward the abutments and in the manner following the theoretical design of a trussed girder. The vertical members, which are under compression, are of plain concrete without reinforcement.

In Fig. 14 (*b*) are shown in cross-section two girders *b* supporting a beam *a*. At *c*, between the ends of the beams, a space 5 inches wide extends along the girders. This space is filled with concrete into which is embedded a reinforcing rod *d*. The purpose of this construction is to tie the columns together in the direction across the building. The slab beams are 12 inches in width, and are reinforced in the top with three ¼-inch rods and in the bottom with three ⅜-inch rods. The web reinforcement in these beams is very light and consists of round wire.

MACHINE-SHOP CONSTRUCTION

16. In Fig. 15 is illustrated a working drawing that shows a section through a building constructed entirely of reinforced concrete. This type of building is particularly suitable for machine shops, foundries, and manufacturing plants of this character.

As will be observed, the center bay is open from the ground floor to the roof, and as the building is of considerable width, this portion is lighted and ventilated by a lantern *a* with pivoted sash along the side. Frequently Star ventilators, as shown at *b*, are placed along the ridge of the lantern, so as to provide ventilation in winter when the side sash are closed. Such ventilators also tend to take care of the heated air that rises to the peak of the lantern.

The wall piers *c*, as can be seen from the size indicated, are somewhat larger than the interior columns. They provide space in each bay for triple windows. The floorbeams *k* of the gallery extend parallel with the walls, so that the soffit of the lintels is close to the bottom of the floor slab. In this way, a maximum amount of light is gained from the windows on the side of the building, both for the ground floor and for the gallery. The gallery floors in buildings of this type are usually designed to sustain from 150 to 250 pounds of superimposed load upon each square foot of surface.

17. The main feature of the design is the arrangement of the central columns, which are provided with brackets *d* to sustain the runway girder *e* for the support of the traveling crane. Ordinarily, it is necessary to carry upon the runway girder a 10- or 12-ton crane with wheel loads of about 15,000 pounds, the wheels being on centers ordinarily about 6 feet apart. With 16-foot bays, 18-inch I beams must generally be used for the runway girders, as in Fig. 15. The top of the runway girder must be reinforced by means of a channel iron *f*, in order to prevent the runway girder from buckling sidewise and thus allowing the wheels of the crane to drop from the track. Considerable care must be exercised in the design of the fastenings used to secure the runway girder to the concrete work, and one of the best means consists in providing through bolts *g* with pipe separators *l*. The holes in the concrete for the through bolts are provided by accurately placing in the forms pieces of pipe of sufficient length. When the runway girder has been set and is secured with bolts, the space between the girder and the column is filled with concrete. In this way, the runway girder is firmly secured and supported in a lateral direction, and no amount of side strain can cause the crane to drop.

From the fact that the runway girder is supported at some distance from the center line of the column, there is a tendency to place upon the column an eccentric load. This tendency is obviated to a considerable extent by making the column of the first story wider, so as to extend under the runway girder. This construction not only increases the strength of the column, but allows the projection of the bracket to be reduced to a minimum. It is well in arranging the reinforcement of the column *h* to place the reinforcing rods *m* in the manner indicated in Fig. 16. In this way is provided additional reinforcement to resist the transverse, or bending, stress produced in the column by the eccentric load.

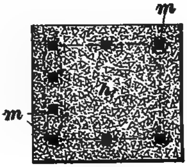
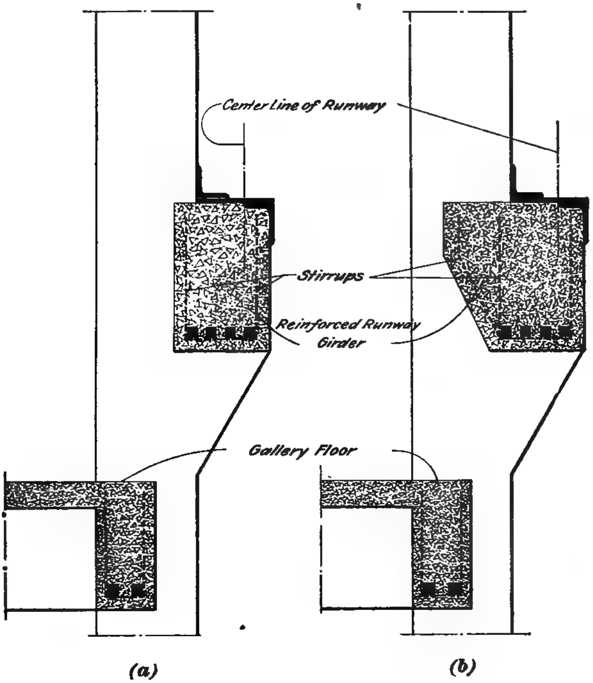


FIG. 16



(a) (b)

FIG. 17

18. In the design of a building of the construction shown in Fig. 15, it is usually necessary to have a traveling crane to serve the gallery. For this reason, the structure should be of sufficient height to allow headroom beneath the lower flange of the runway girder. Also, in order to assist in landing goods upon the gallery, platforms should be projected at intervals, as shown at *i*. These platforms can be made of a length equal to one bay, and can be constructed by forming a cantilever beam opposite the ends of the principal girder of the balcony. The cantilevers can be reinforced by extending the reinforcing rods of the girder; and when arranged with stirrups, as shown in the figure, such rods will sustain any load that will likely be placed upon them.

If it is not required to make the lantern the entire width of the central bay, this structure can be carried upon cantilevers formed by extending the roof girders beyond the interior columns and reinforcing them on the top.

If it is not possible to obtain the structural-steel work required for the runway girder in time, the girder may be constructed of reinforced concrete, as shown in Fig. 17 (*a*). The greatest difficulty encountered in designing the girder of reinforced concrete consists in so proportioning it as to provide sufficient compressive strength in the upper portion of the section. Sometimes it is possible to enlarge the section of the girder toward the top, as shown in (*b*), and thus gain additional compressive strength as well as greater strength sidewise.

REINFORCED CONCRETE USED FOR RECONSTRUCTION

19. Reinforced-concrete construction may be used successfully to reinforce old buildings that are to be altered to carry heavy loads or buildings that have shown weakness in construction due to age, vibration, or overloading. In Fig. 18 is shown how a reinforced-concrete frame and floor system may be introduced in buildings originally constructed with wooden joists, or timber girders, sustaining a wooden floor. In this instance, the walls of the buildings were found to be good and straight and capable of sustaining their own weight, but they were not deemed sufficiently strong to carry the dead load of the concrete floors and the live loads that these floors were designed to sustain. It was therefore proposed to

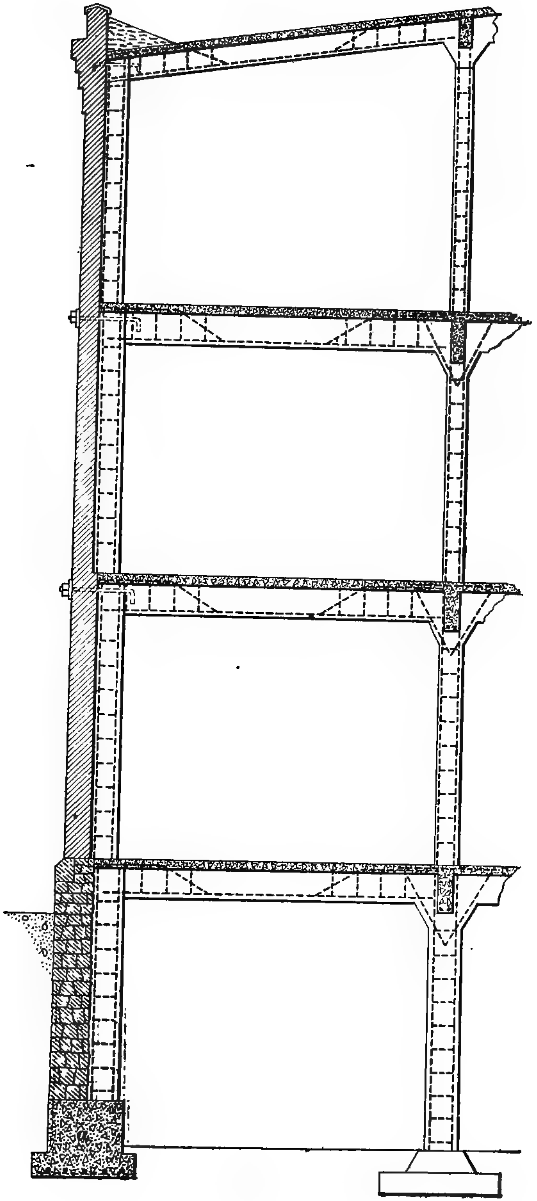


Fig. 18

provide inside the building a complete reinforced-concrete construction consisting of wall piers and a central row of reinforced-concrete columns supporting beams, girders, and floor slabs of the same construction. In this manner, the brick walls of the building were relieved of all the weight of the new floor construction.

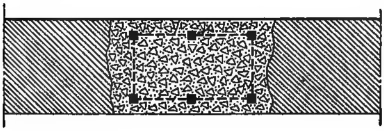


Fig. 19

In forming the concrete wall piers, the brickwork was used as one side of the form and the concrete of the wall piers was placed directly against it. In order to tie the walls to the new concrete work, iron anchors were placed through the walls at the level of each floor and embedded in the concrete as the work progressed. After shoring the wall, the necessary foundations were provided by underpinning, as indicated at *a*. The floor system of this construction was reinforced in the usual manner, as suggested in the illustration.

20. Sometimes it is not advisable to provide wall piers that occupy considerable room on the inside of the building. In cases of this kind, walls may be reinforced by the method illustrated in Fig. 19. In this instance the brick walls were solid and had no pilasters. They were reinforced by cutting vertical openings in them the width of the concrete piers, and then building the piers in these openings by using forms on the faces, as shown in Fig. 20. In this manner, piers sufficiently heavy to carry trusses for a clear span the full width of the building were introduced.

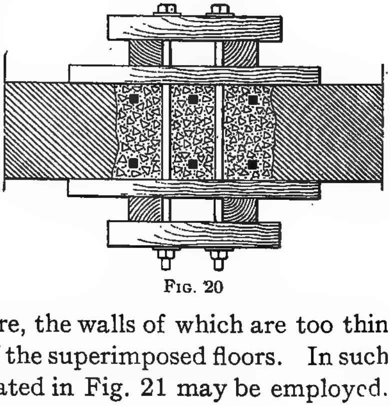


Fig. 20

21. It is sometimes desirable to add several stories to an existing structure, the walls of which are too thin and weak to carry the load of the superimposed floors. In such instances, the method illustrated in Fig. 21 may be employed.

In this figure, the old wall of the existing one-story building is shown at *a*. Over this wall it was proposed to place another story of considerable height, so that the traveling crane could reach all parts of the second story. From this condition, the height of the second floor was fixed at 22 feet, and it was found that the wall *a* was entirely inadequate to carry the combined weight of the second floor and the concrete roof and was totally unfit to sustain the crane load as required.

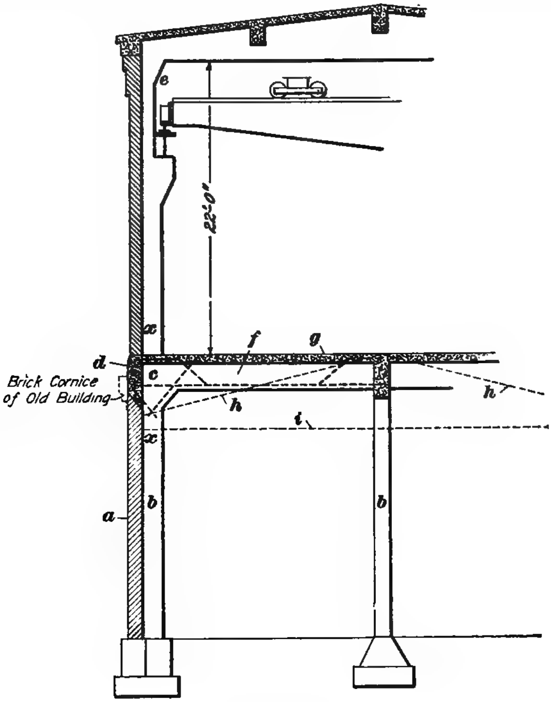


FIG 21

In carrying out the work, the reinforced-concrete construction was arranged with wall piers and concrete columns, as indicated at *b*, and the walls were underpinned at the points adjacent to the wall piers in order to provide for the footings of these members. The outside wall of the building was not disturbed except along the eave line, where the cornice was taken down and replaced with a reinforced-concrete beam *d*. As will be observed, this beam is supported at each wall pier by a cantilever *c* extending from the pier *b* and having reinforcing rods that extend from the girder *f* through the pier into the cantilever. This arrangement makes the floor slab *g*, girder *f*, pier *b*, beam *d*, and cantilever *c* monolithic. The girder *d* carries the weight of the spandrel wall in the second floor, but the roof and the crane loads are transferred to the wall column on the floor below by means of a reinforced-concrete column. The dotted lines *h* and *i* represent the roof construction of the old building.

From the fact that the crane spans the entire width of the building, the roof members had to be made sufficiently strong to carry over this span, and in order to brace the building near the roof, the heavy concrete brackets *e* had to be introduced and placed in such a way as not to interfere with the mechanism of the traveling crane.

22. In designing the cantilevers for constructions similar to that just described, great care must be taken to see that there is sufficient shearing resistance along the plane *x x*; besides, sufficient steel must be introduced not only to take care of the transverse stress, but to provide the necessary shearing resistance as well. The cantilever beams should also be made of considerable width—not less than 18 inches—in order that the rods in the cantilever may be so spaced as to obtain a sufficient mass of concrete around them. Sometimes the cantilever may be made square at the bottom, as shown by the dotted line, so that if there is any tendency to failure, the weight will come upon the brick wall, which, though it might be overloaded, would save the building from sudden collapse.

Constructions of this kind may be carried on without interrupting the operations in the building in which the changes are being made. This method, however, results in slow and costly work, and should only be employed when it is impossible for the owner to vacate for a sufficient length of time to finish the building without interference.

AREA AND VAULT WALLS

23. In conjunction with reinforced-concrete buildings, it is customary to make the area and vault walls of the same material. If the depth of the basement floor below the sidewalk is from 10 to 15 feet, such walls should be carefully designed in order to resist the thrust due to the pressure of the earth.

If area or vault walls extend between the reinforced-concrete columns of the building, they may be made from 8 to 12 inches in thickness and may be reinforced for transverse stress laterally between the columns. This construction is shown in Fig. 22 (*a*), and, as will be observed, the reinforcing rods *a* are placed on the inside surface because this is the tension side of the wall. In order to attain economy in the design of such walls, rods can be placed close together near the bottom of the wall, the distance between them increasing toward the top of the wall as the thrust from the earth decreases. In addition to the reinforcing rods, shrinkage rods *b* of ⅜-inch round or

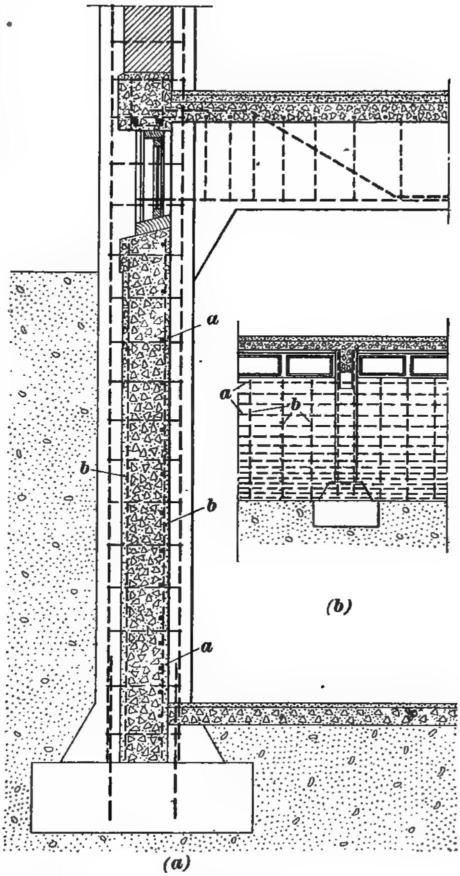


FIG. 22

$\frac{5}{16}$-inch square twisted bars should be inserted vertically throughout the wall.

As walls of this kind are frequently put in after the construction of the building has advanced, it is not always possible to extend the reinforcing rods *a* into the column, as shown in Fig. 22 (*b*). In such cases, shear rods should be placed through the columns and allowed to project, or else the columns should be constructed with grooves in the sides so that the wall will bond in.

24. In order to gain all the available space in the basement, it is customary to extend the basement out underneath the sidewalk, so that a vault is formed in the manner shown in Fig. 23. Here, the stability of the retaining wall of the vault is not assisted by the concrete columns of the building, and the wall must therefore be provided with buttresses *a* of reinforced concrete. These buttresses act as beams in resisting the thrust of the earth, and are incorporated with the concrete beams *b* that support the pavement. They are designed and reinforced sufficiently to resist the pressure of the earth on the concrete wall placed between. This wall

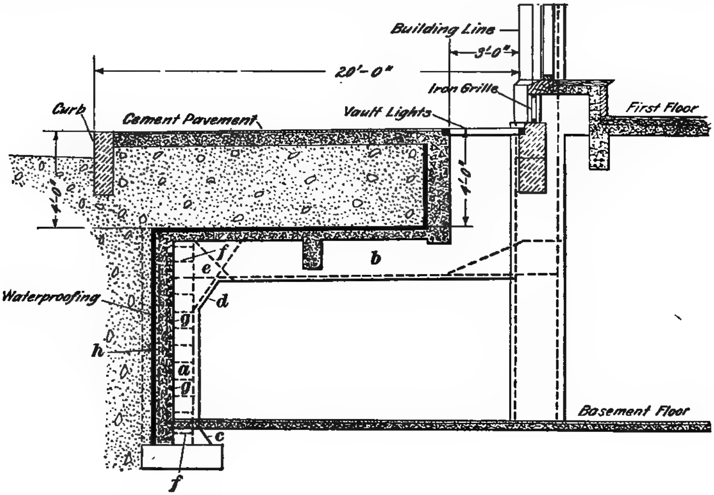


Fig. 23

may be made from 8 to 12 inches in thickness, depending on its height and the distance from center to center of buttresses. In forming the buttresses, the span subjected to transverse stress is considerably reduced by flaring them at the bottom, as at *c*, and by forming a bracket between the buttress and the beam, as at *d*. The main reinforcing rods of the beams and buttresses may be arranged by bending, so as to interlace and also reinforce the brackets, as shown at *e*. It is well to use some stirrups *f* with the reinforcing rods, especially where the wall is high, so as to produce great transverse stress on the concrete buttresses. The wall between the buttresses is reinforced with rods *g*, which should extend through the buttresses where it is possible to arrange them in this manner; besides, there should be an ample number of shrinkage rods *h* extending vertically.

SPRINKLER TANKS

25. In nearly all buildings of an industrial or commercial nature there are installed sprinkler and fire-protection systems that require large water tanks. The requirements of the fire-insurance underwriters generally demand that the tanks for sprinkler systems be supported at a distance of at least 20 feet above the highest point of the roof. In capacity, these tanks range from 15,000 to 40,000 gallons, so that the weight to be held up is very great. In order to provide a support for such tanks, the common practice in reinforced-concrete structures is to run four of the columns up above the roof. These columns are then connected and upon them a reinforced-concrete platform is built. Such columns are made larger than the normal columns or they are more strongly reinforced, so that a greater unit stress may be used in their calculation.

A typical reinforced-concrete tank tower *f* for a commercial building is shown in elevation in Fig. 24 and in plan in Fig. 25. In Fig. 24, the columns supporting the tank are of the sections indicated at *a*. This section is adopted in order that the tower will be stiff enough to resist wind pressure. The projections *h* of the returns are carried on the brackets *i* beneath the girders and beams of the roof construction. Usually, there is sufficient room beneath the sprinkler tank for a house supply tank *g*, so that a tier of beams and girders is introduced, as at *b*, that greatly stiffens the tank tower. The top of the tower is formed with beams, and heavy girders *c* connect the columns on the four sides. The girders are usually arranged with strong brackets that connect the returns of the columns, or else a semicircular arch *k* is used at this point, as shown. In order to provide a walk, or passageway, around the tank for painting and inspection, the slab at the top of the tank is extended and supported on cantilever brackets *d*.

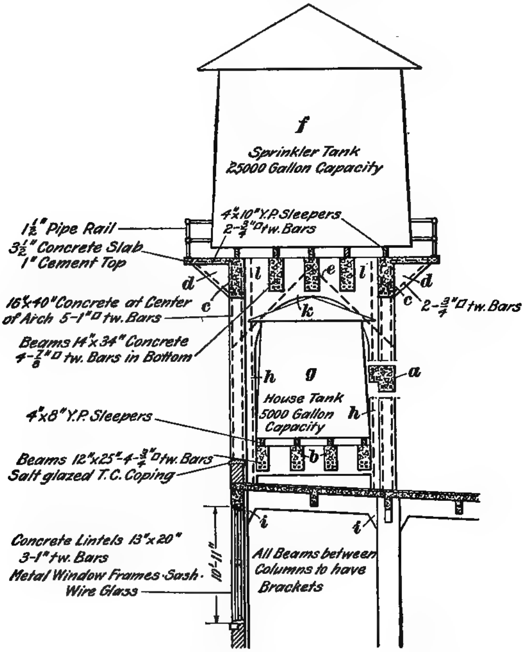


FIG. 24

26. In some instances, as in the construction shown in Fig. 24, the beams that support the sprinkler tank are made of rectangular section; that is, the slab is not run entirely across the top of the tank tower. In this way there are provided between the beams openings *l*, Fig. 25, that tend to create circulation of air around the tank. When this construction is used, however, the beams become very deep; they must therefore be reinforced in the top as well as in the bottom.

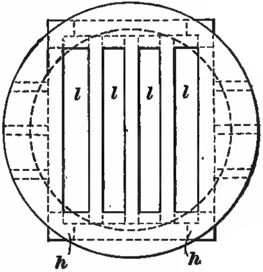


FIG. 25

Ordinarily, reinforced-concrete tank towers are not calculated for wind pressure, the column supports being made to sustain directly only the dead load due to the tank filled with water.* If there is any doubt regarding the stability of such a tower to resist wind pressure, the usual calculations can be made to determine the increased compressive stress on the leeward column. All the necessary reinforcement for the construction of a typical tank tower is illustrated in Fig. 24. The girders and beams at the top of the tower, as well as those supporting the house supply tank, should be adequately reinforced. The reinforcement should interlace, and the construction should be made rigid by introducing reinforcement at the angle brackets, as shown at *e*.

SAW-TOOTH SKYLIGHT CONSTRUCTION

27. Saw-tooth skylights derive their name from the similarity of their contour to the tooth of a saw. They are extensively used for lighting the top floors of industrial buildings. The purpose of the saw-tooth skylight, an example of which is shown in Fig. 26, is to provide a lantern with an oblique face *a*, that is glazed throughout, is raised above the roof, and is usually facing north, so as to get the maximum amount of daylight without the glare of sunlight. The general contour of these skylights is shown in the figure. As will be observed, the angle of slope of the face *a* is about 15° from the vertical, and is arranged at this angle so that, in temperate zones, the sun will never approach the meridian sufficiently to throw sunlight in the building. The angle of slope *s* of the roof is fixed by the width *w* of the skylight and the height *h*. The angle *s* must, however, in all cases be less than about 35° if the roof is to be covered with a slag or gravel.

In view (*a*) is indicated a skylight spaced so as to cover an entire bay extending from column to column. It is not necessary, however, to arrange the skylights in this manner. They can be placed in any position on the roof by providing cross-beams *c*, as shown in (*b*), and starting the face and roof slope of the skylight from these beams.

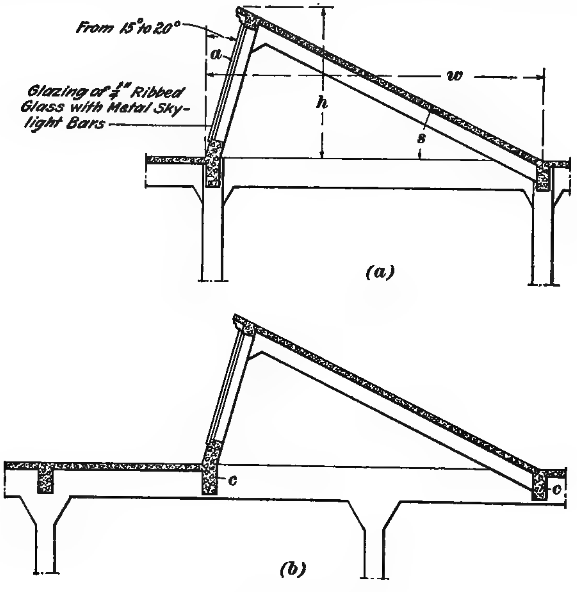


FIG. 26

28. It is of advantage in saw-tooth skylight construction to use as few beams and girders beneath the skylight as possible, so that there will not be any unnecessary shadows cast in the room below. Owing to structural considerations, however, the building must be tied well together in both directions, and for this reason some connecting girders and beams must be employed.

The best framing for saw-tooth skylights is that shown in Fig. 27, which illustrates diagrammatically the framing plan and section of a typical construction. Plans of this kind are not drawn in conformance to ordinary drawing rules. For example, as shown in the figure, some of the parts, such as the girders, beams, and rafters, may be indicated by single dotted lines, while other parts, as columns and skylight supports, may be shown in cross-section.

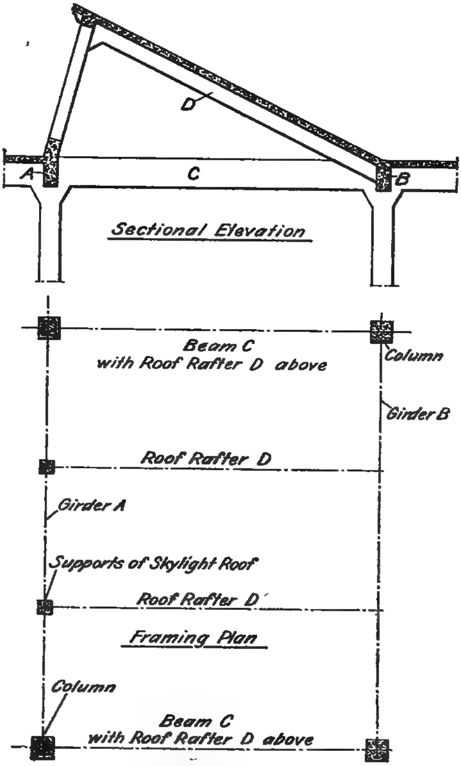
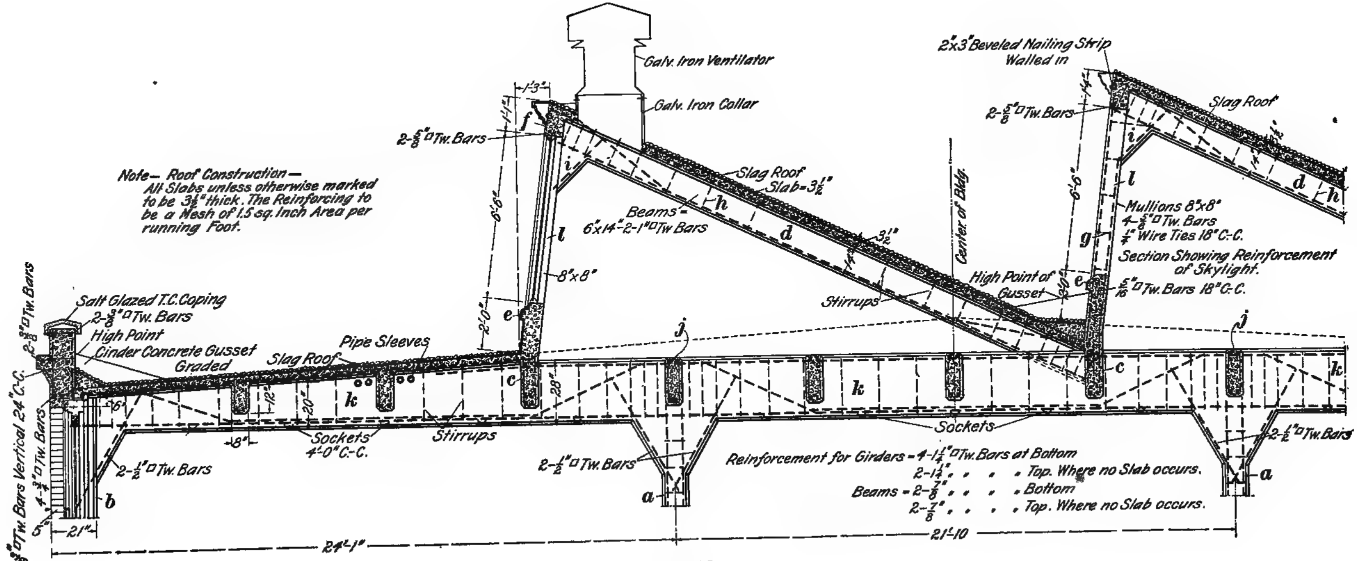


FIG. 27

29. A construction drawing of a reinforced-concrete saw-tooth skylight is shown in Fig. 28. As shown in this illustration, the columns *a* support the roof construction. The outside wall of the building is indicated at *b*. The skylights are set back some distance from the parapet wall, and are supported upon beams *c* bearing upon the transverse girders *k*. The construction of the skylight consists of the lintel *f* and rafter beams, as at *d*, supported upon 8″×8″ mullions *l*, which are monolithic with the curb *e*. The curb is well reinforced with shrinkage rods, and the mullions are reinforced with four $\frac{5}{8}$-inch rods tied at intervals with wire ties, as at *g*. The rafter beams are reinforced in the usual way and are well provided with stirrups, as shown at *h*.


Galv. Iron Ventilator
Galv. Iron Collar
2"x3" Beveled Nailing Strip Walled in
Slag Roof
Slag Roof Slab=3$\frac{1}{2}$"
Beams= 6"x14"-2-1"□Tw Bars
2-$\frac{5}{8}$"□Tw. Bars
2-$\frac{5}{8}$"□Tw. Bars
1'-3"
1'-1"
1'-4"
6'-6"
6'-6"
2'-0"
3'-0"
8"x8"
Mullions 8"x8"
4-$\frac{5}{8}$"□Tw. Bars
$\frac{1}{4}$" Wire Ties 18" C-C.
Section Showing Reinforcement of Skylight.
$\frac{5}{16}$"□Tw. Bars 18" C-C.
High Point of Gusset
Center of Bldg.
Stirrups
3$\frac{1}{2}$"
Salt Glazed T.C. Coping
2-$\frac{3}{8}$"□Tw. Bars
2-$\frac{3}{8}$"□Tw. Bars
High Point
Cinder Concrete Gusset Graded
Slag Roof
Pipe Sleeves
$\frac{3}{8}$"□Tw. Bars Vertical 24" C-C.
4-$\frac{3}{4}$"□Tw. Bars
2-$\frac{1}{2}$"□Tw. Bars
2-$\frac{1}{2}$"□Tw. Bars
2-$\frac{1}{2}$"□Tw. Bars
Sockets 4'-0" C-C.
Sockets
Stirrups
6"
8"
12"
20"
28"
5"
21"
24'-1"
21'-10
a
b
c
d
e
f
g
h
i
j
k
l

Note— Roof Construction—
All Slabs unless otherwise marked to be 3$\frac{1}{2}$" thick. The Reinforcing to be a Mesh of 1.5 sq. Inch Area per running Foot.

Reinforcement for Girders = 4-1$\frac{1}{4}$"□Tw. Bars at Bottom
2-1$\frac{1}{4}$" " " " Top. Where no Slab occurs.
Beams = 2-$\frac{7}{8}$" " " " Bottom
2-$\frac{7}{8}$" " " " Top. Where no Slab occurs.

FIG. 28

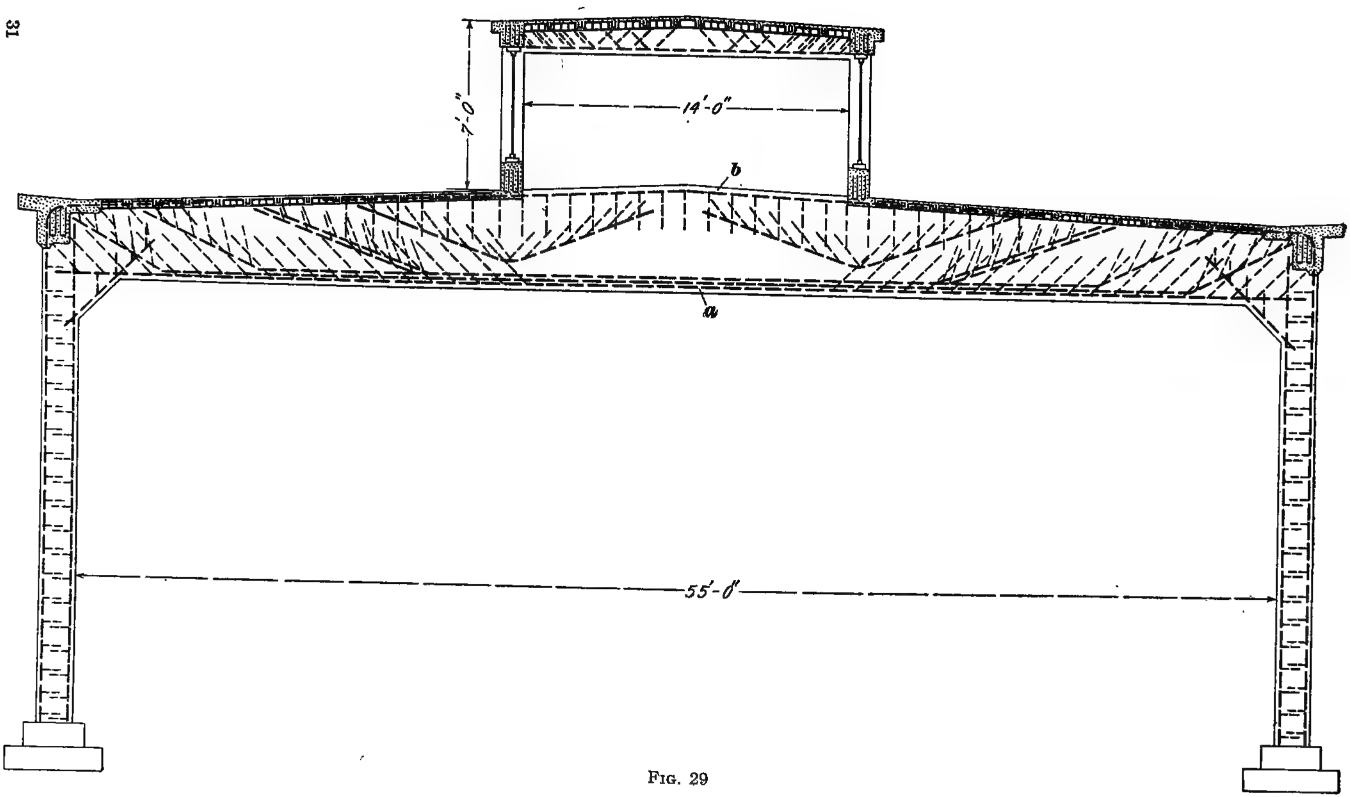


Fig. 29

It will also be observed that brackets *i* are formed between the rafter members and the mullions of the skylight construction. These brackets greatly stiffen and strengthen the construction at this junction. The cross-girders, as at *j*, are frequently omitted, but by their use the roof construction is strengthened and the building is stiffened laterally at the roof line.

The other features of the construction, as well as the reinforcement throughout, are clearly shown in the figure. The reinforcement rods are carefully interlaced to insure that the connections and junctions are well tied together. This matter should always receive careful attention.

REINFORCED-CONCRETE ROOF TRUSSES AND GIRDERS

30. Roof trusses, or girders, are sometimes constructed of reinforced concrete. Such members are very seldom made with separate chords and web members, because it is usually more convenient to mold the entire member solid, the reinforcement being placed so as to occupy the position of the top and bottom chords and the web members.

31. In Fig. 29 is shown a reinforced-concrete roof truss, or girder, having a span of 55 feet between supports. The entire construction is arranged for the use of Kahn truss bars, and the roof slab consists of hollow tile with concrete joists. Naturally, the bulk of the reinforcement is in the lower part *a*, or tension side, of the girder. The upper chord is reinforced toward the center at *b* in order to provide additional compressive resistance across the lantern, where the truss does not have the advantage of the **T** section of concrete formed by the slab and the girder.

As will be observed, the chord members are bent up toward the supports, and, by the use of the prongs, the concrete work is well stirruped. It will likewise be observed that the stirrups projecting downwards from the upper chord reinforcement are numerous. These stirrups interlace with

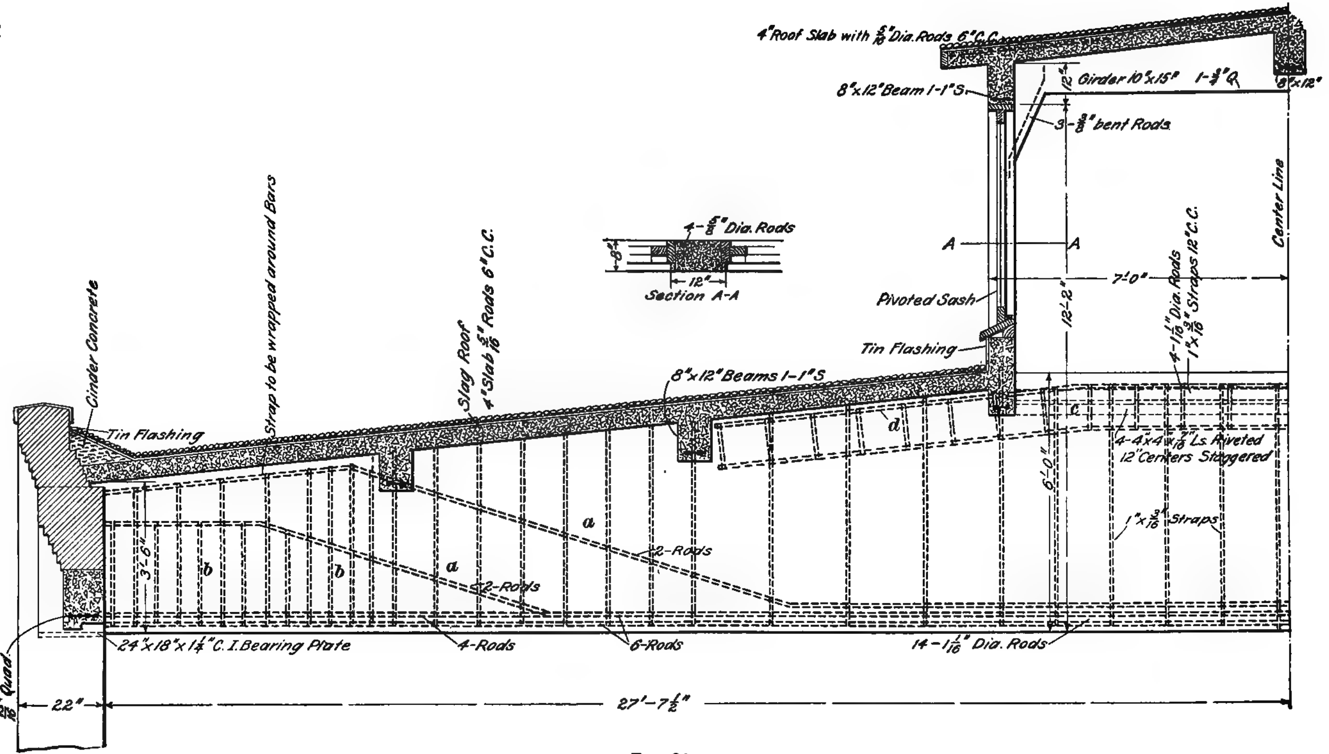

4" Roof Slab with 5/16" Dia. Rods 6" C.C.
Girder 10"x15" 1-3/4" Q
8"x12"
12"
8"x12" Beam 1-1" S
3-3/8" bent Rods
Center Line
4-5/8" Dia. Rods
8"
12"
Section A-A
A
A
7'-0"
12'-2"
Pivoted Sash
Tin Flashing
4-1 1/16" Dia. Rods
1"x3/16" Straps 12" C.C.
Cinder Concrete
Strap to be wrapped around Bars
Slag Roof
4" Slab 5/16" Rods 6" C.C.
8"x12" Beams 1-1" S
Tin Flashing
d
c
6'-0"
4-4x4x7/16 Ls Riveted
12" Centers Staggered
a
2-Rods
1"x3/16" Straps
3'-6"
b
b
a
2-Rods
13/16" Quad
24"x18"x1 1/4" C.I. Bearing Plate
4-Rods
6-Rods
14-1 1/16" Dia. Rods
22"
27'-7 1/2"

Fig 30

the stirrups of the web reinforcement and thus greatly strengthen the truss beneath the concentrated load due to the weight transmitted by the sides of the lantern.

32. A construction similar to that just described is shown in Fig. 30. In this case, the span is about the same, being 55 feet 3 inches. This concrete truss, or girder, is reinforced with round rods, the bottom chord of the truss containing fourteen $1\frac{1}{16}$-inch round rods. Ten of these rods run entirely through to the abutment in a straight line, and the other rods *a* are bent up in the form of a camber rod, or truss beam. Numerous stirrups *b*, composed of $\frac{3}{16}'' \times 1''$ bar or strap iron, are provided at close intervals near the abutments. These stirrups are spread farther apart as the center of the span is approached. As the truss is rectangular in section inside the lantern and is deprived of the advantage of the **T** section of the concrete at this point, it must be reinforced for compression by embedding in the concrete a structural shape *c* built up of angles in the form of a star section, as shown in detail in Fig. 31. Surrounding this star section and parallel to it are four $1\frac{1}{16}$-inch rods *d*. These rods are well tied together at close intervals with $1'' \times \frac{3}{16}''$ strap-iron ties, so that the entire arrangement makes the top of the girder very strong in compression. The metal in this portion may be figured for at least 10,000 pounds to the square inch in compression. The lantern is supported upon reinforced-concrete uprights, as shown in Fig. 30. The construction of the lantern, as well as its reinforcement, is clearly indicated in the illustration.

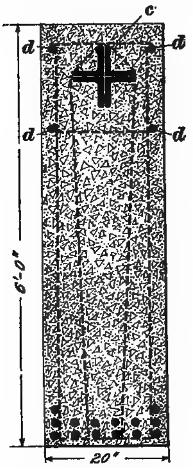


Fig. 31

The cost of this type of reinforced-concrete roof truss is less than a structural-steel truss and the necessary fireproofing for the same, but is somewhat more than the cost of a wood truss made up of $10'' \times 10''$ or $10'' \times 12''$ yellow-pine timbers.

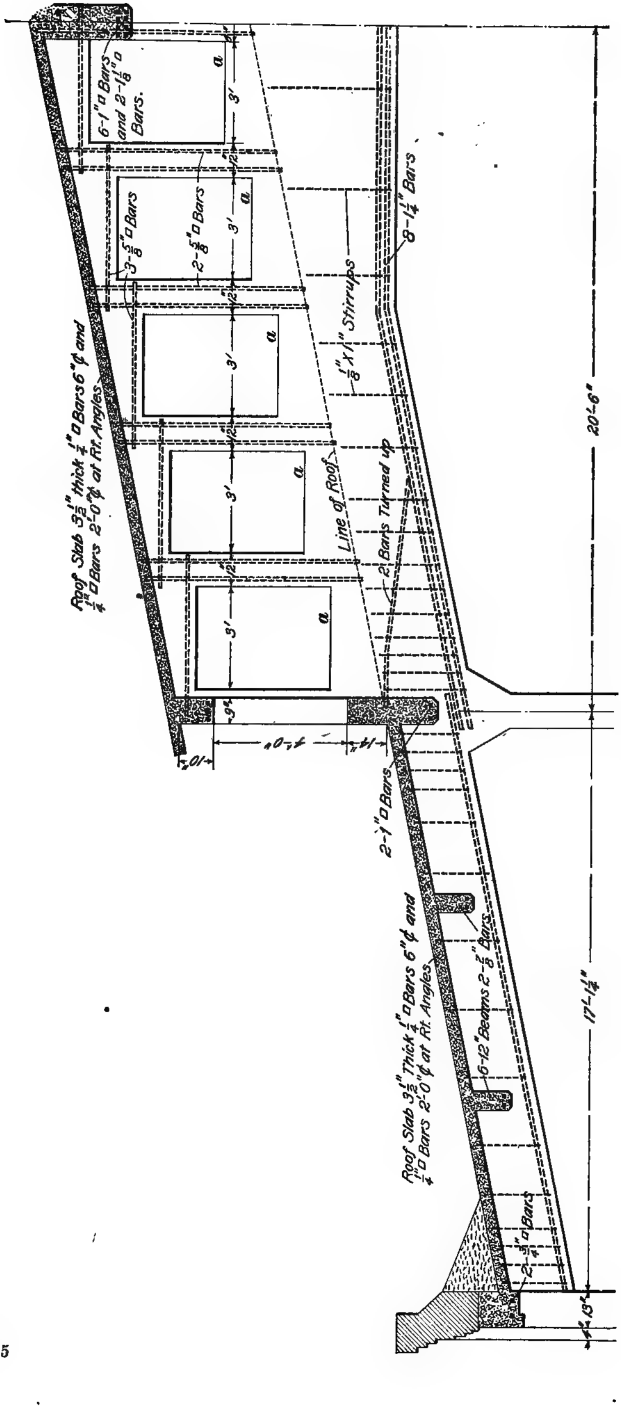

Roof Slab 3½" thick ¼" ▫ Bars 6" ¢ and ¼" ▫ Bars 2'-0" ¢ at Rt. Angles
6-1" ▫ Bars and 2-1⅛" ▫ Bars.
3-⅝" ▫ Bars
2-⅝" ▫ Bars
a
3'
12"
Line of Roof
⅛" x 1" Stirrups
8-1¼" Bars
2 Bars Turned up
20'-6"
10"
4'-0"
14"
9"
2-1" ▫ Bars
Roof Slab 3½" Thick ¼" ▫ Bars 6" ¢ and ¼" ▫ Bars 2'-0" ¢ at Rt. Angles
6-12" Beams 2-⅞" Bars
2-¾" ▫ Bars
17'-1½"
13"

FIG. 32

33. Another form of reinforced-concrete construction for a large roof span is shown in Fig. 32. Here, the building has a 41-foot span between centers of columns, and this span carries a lantern. As in the previous cases, the reinforced-concrete truss is deprived of the advantage of the **T** section formed by the concrete roof slab. The top portion of the concrete girder requires no reinforcement, however, because the depth is great enough to prevent overstressing the concrete at the top of the section, the girder extending upwards toward the window sills *a*. The fact that the lower chord of the girder has a slanting position does not interfere with the usual calculations, as the bending moment and the resistance at any section can be determined by the usual formulas.

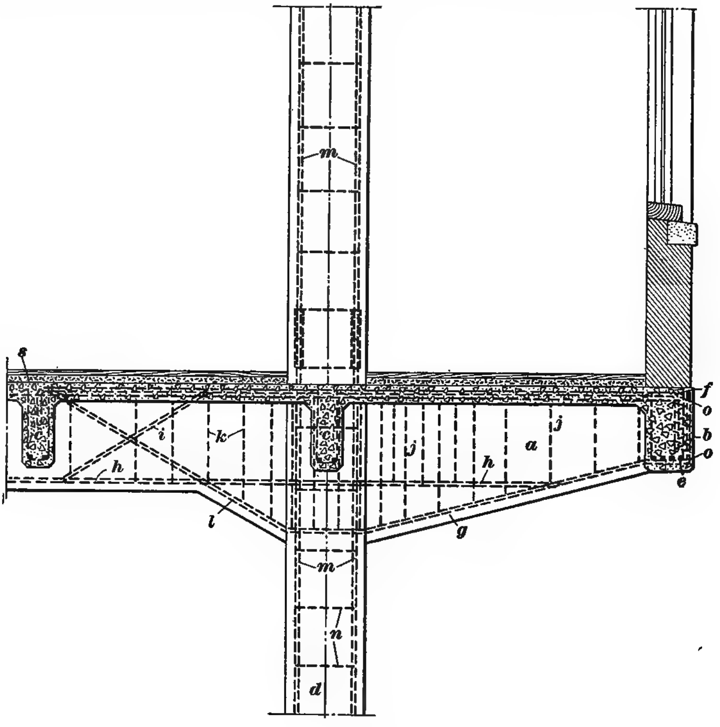


FIG. 33

It is well in all constructions of this kind to provide ample stirrups and to bend the reinforcing rods in the lower chord, where they are not needed, in such a manner as to cross the lines of possible fracture due to the combined tension and shear.

34. Cantilevers.—It is not unusual in buildings to use reinforced cantilever beams to support overhanging balconies. Such beams are also used to carry the walls of buildings over driveways, etc.

In Fig. 33 is shown the construction at the rear portion of a commercial building. The rear wall of the building is supported at each story by cantilever beams *a*, which are monolithic with the columns and floor construction. The walls, which are supported at the end of such cantilever beams, are made as light in weight as possible, though frequently 13-inch brick walls supported upon reinforced-concrete beams *b* extending through the ends of the cantilevers are used as curtain walls. These beams are necessarily made the same width as the wall to be supported, so that none of the weight will come upon the floor slab. The cantilever beams are generally an extension of the floor girders, the floorbeams *c* extending into the columns and girders. In high buildings, the reinforced column *d* would contain a steel core, but the column shown in the figure is reinforced with longitudinal steel rods *m* tied together with wire tires *n* at intervals. The girder supporting the outer wall is reinforced in the manner shown at *e*, and the cantilever is reinforced with both top and bottom rods, as shown at *f* and *g*. The rods *g* generally extend under the reinforcement *e*, so that the latter will take a bearing upon them, and the top and bottom rods are bent as at *o*, so that they can be wired together at the end of the cantilever. The top rods of the cantilever are under tension, and as the beam is weak under compression, being rectangular in section at the lower part, the rods *g* must be provided to furnish additional compressive resistance. These rods extend through the column and up into the girder of the floor system, as at *l*, in this way transmitting the reactions from the cantilever into the floor construction. Some of the bottom rods *h* of the girder should be carried through into the cantilever and form part of the reinforcement of this member. The truss rod *i* of the girder of the floor system should, where practicable, be carried the full length of the top of the cantilever, and the other rods at the top of the cantilever should be carried well back into the floor construction and turned down on the end, as at *s*, so as to get sufficient hold, or bond, in the concrete. The cantilever beam should be well provided with stirrups *j*, which should be more numerous toward the support. The girder of the floor system should likewise be well stirruped adjacent to the column, as shown at *k*. It is usually sufficient to make these stirrups of $\frac{3}{8}$-inch round bars, $\frac{5}{16}$-inch square bars, or $\frac{1}{8}'' \times 1''$ flat bars.

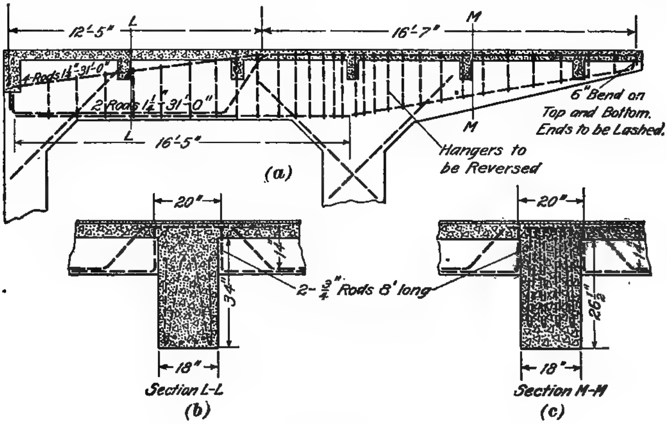


FIG. 34

35. A method of placing the reinforcement that differs slightly from the one just described is shown in Fig. 34. The reinforcement is shown in side elevation in (*a*), while sections through the girder and the cantilever are shown in (*b*) and (*c*), respectively. These sections also indicate the method of placing the stirrups. The cantilever, besides being calculated for the bending stresses, should always be investigated for shearing stresses adjacent to the column and throughout, provided there is any doubt about its ability to resist these stresses.

As a rule, cantilever beams are used to a limited extent in building construction. They should be designed with ample factors of safety and should be reinforced with more care than the simple beams or girders throughout the floor construction, for these latter have the advantage of continuity, and are actually stronger because of this continuity than is shown by theoretical formulas.

DOME CONSTRUCTION

36. Reinforced concrete is a convenient material to use for the construction of domes and vaults. In many instances, the main supporting members of the dome or vault are made of structural steel and the shape of the dome, or sheathing, is worked out in reinforced concrete. Large domes have also been constructed by building up previously molded and properly shaped concrete blocks in concentric walls and filling the space between with a cinder concrete, which material has the advantage of being light in weight and sufficiently strong for the purpose. The best construction, however, if an opening is to be left for a lantern, or skylight, is to use a properly reinforced concrete shell with a heavy ring at the spring of the dome and around the top. Under most domes is arranged a ceiling dome. This dome is placed some distance below the roof dome, and independent of it and can likewise be constructed of reinforced concrete. Where architectural treatment permits, this ceiling dome may be reinforced with ribs. These members strengthen the construction and provide a ground, or base, for working on the plaster moldings and finishings.

37. A typical reinforced-concrete dome construction is illustrated in Fig. 35. Such domes as this are made from 30 to 50 feet in diameter. They must be heavily reinforced against spreading around the perimeter at the spring of the dome, as shown at *a*, and this ring of concrete must also be strongly reinforced. A similar stiffening ring must be provided around the top of the dome, as at *b*, provided a bull's eye, or lantern, is to be used at this place. It is usually convenient to make the dome shell of the same thickness throughout, keeping it as thin as possible so as to lessen the dead weight. The reinforcement of the dome shell should consist of circular hoops *c* of light reinforcing metal placed at close intervals and reinforcing rods *d* running from the top to the bottom of the dome, following the curve. These rods, however, are not so important as the hoop reinforcement. In the illustration, the inside layer *h* of concrete is shown partly broken away in order to lay bare the reinforcing rods. The outside layer *g* is shown in position, and the reinforcing rods are located near the inside of the dome where the concrete is required to sustain tension.

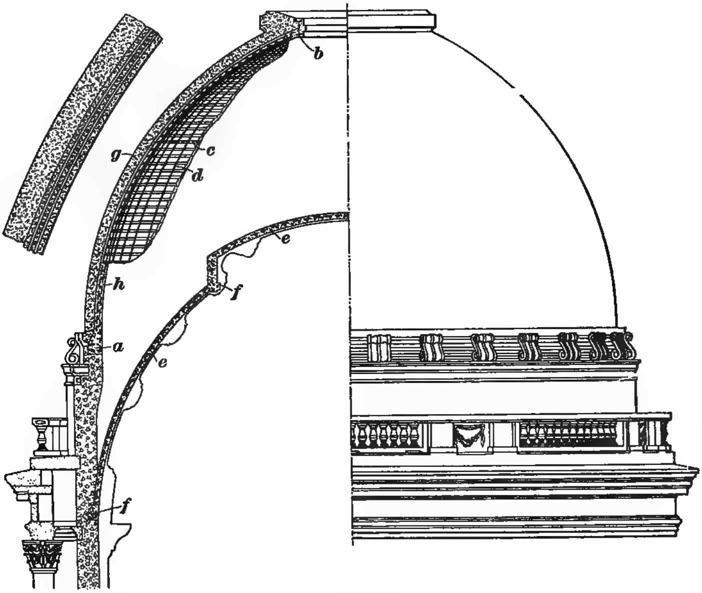
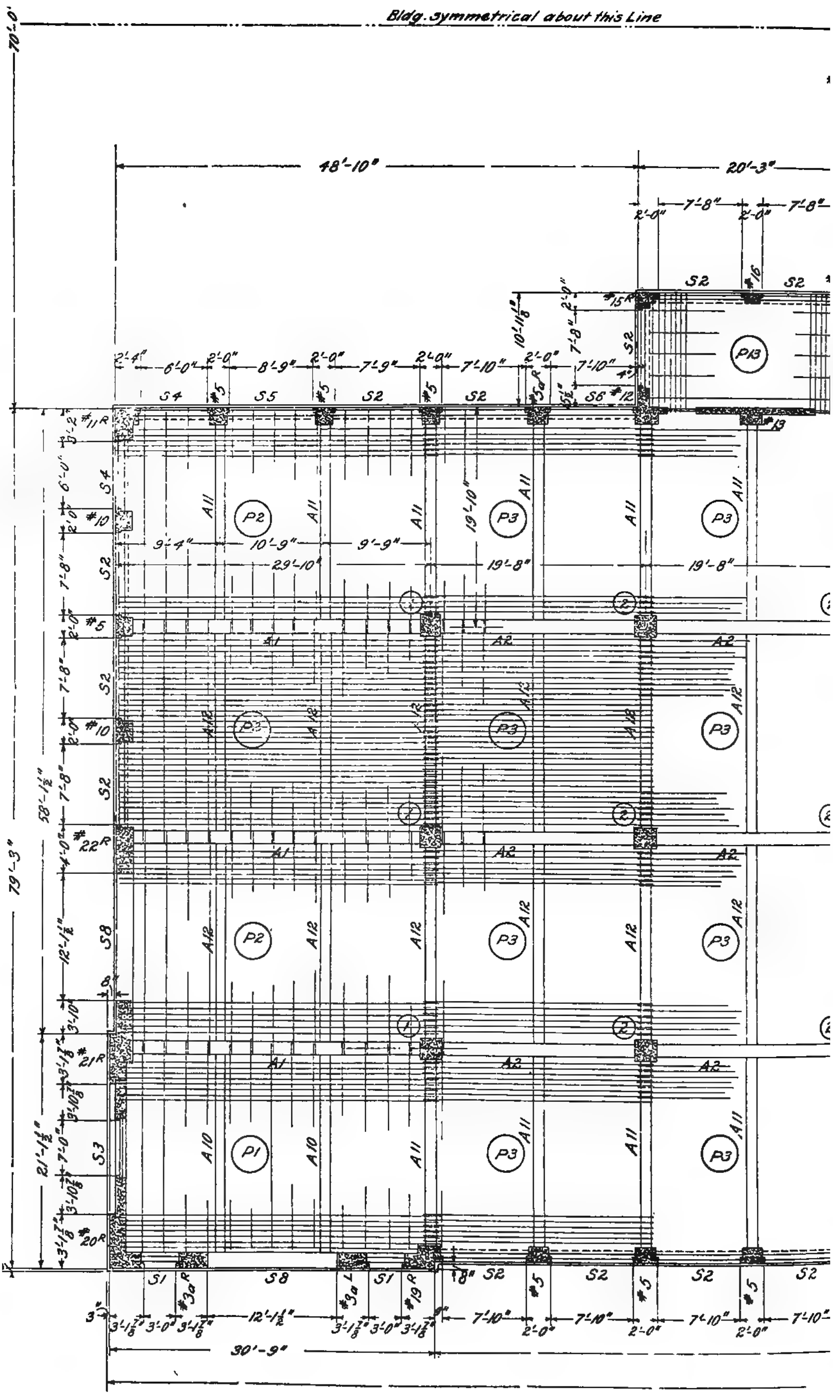
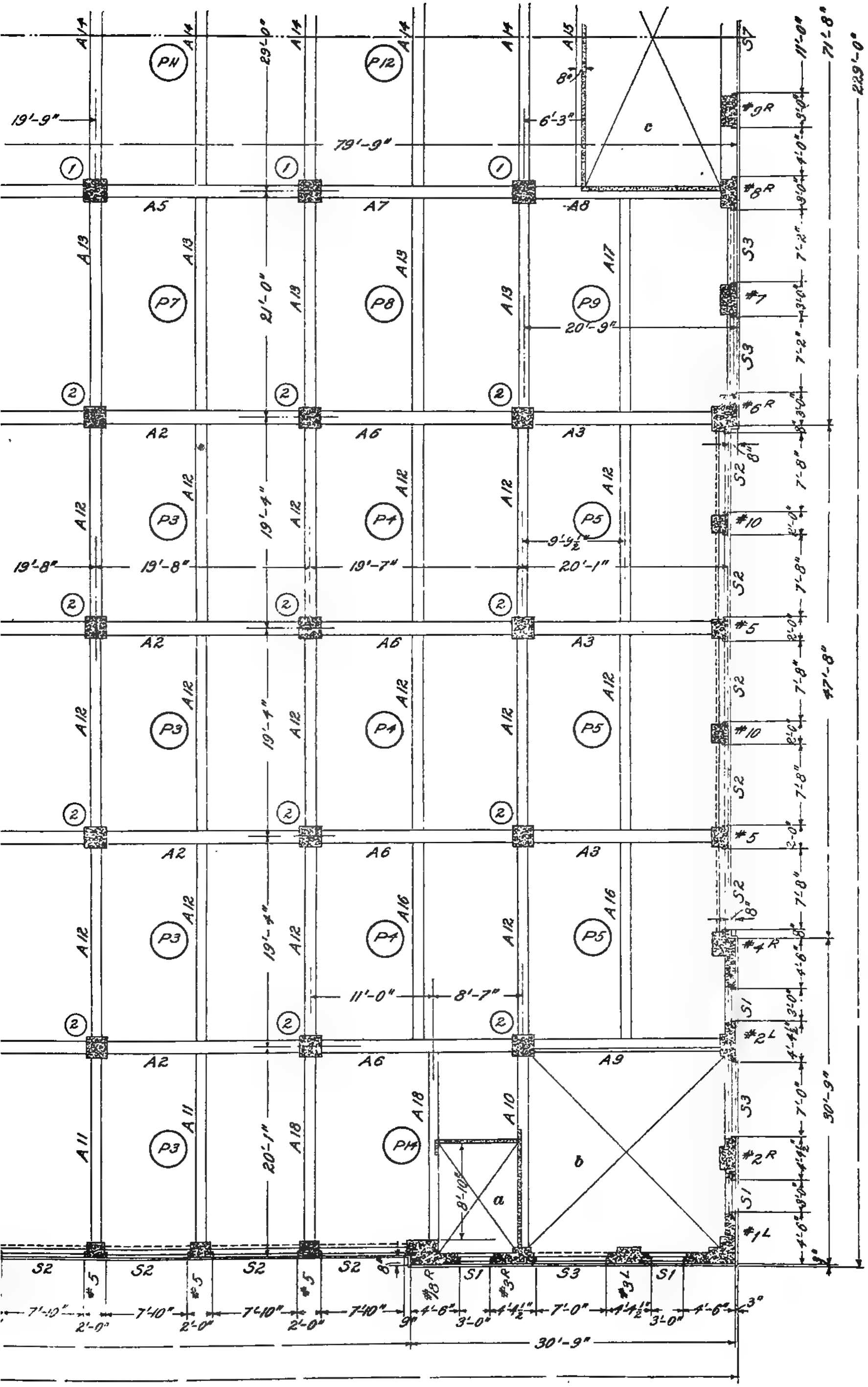
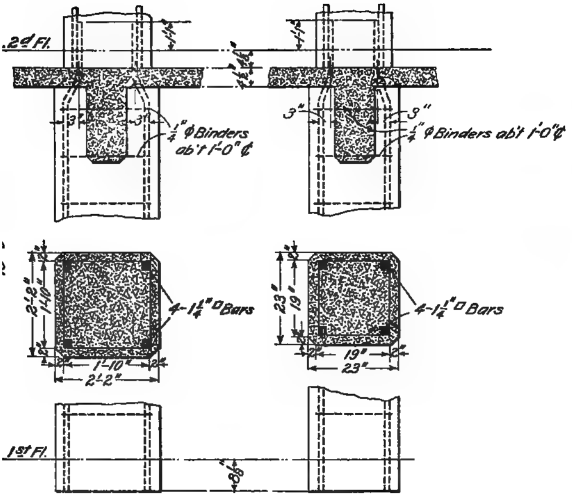


Fig. 35

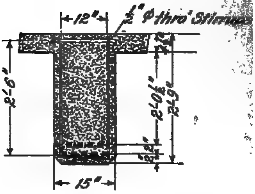
The ceiling dome in the construction illustrated in Fig. 35 is shown at *e*. This dome is reinforced with rings *f* at the bottom and near the top, and, besides, it has reinforcement


Bldg. symmetrical about this Line
70'-0"
48'-10"
20'-3"
2'-0"
7'-8"
2'-0"
7'-8"
S2
#16
S2
#15R
10'-11 1/8"
2'-0"
7'-8"
S2
P13
2'-4"
6'-0"
2'-0"
8'-9"
2'-0"
7'-9"
2'-0"
7'-10"
2'-0"
7'-10"
S4
#5
S5
#5
S2
#5
S2
#3a R
5 1/2"
S6
#12
#11R
#13
3'-2"
6'-0"
S4
2'-0"
#10
A11
P2
A11
A11
19'-10"
P3
A11
A11
P3
A11
9'-4"
10'-9"
9'-9"
29'-10"
19'-8"
19'-8"
7'-8"
S2
2'-0"
#5
1
2
2
A1
A2
A2
7'-8"
S2
2'-0"
#10
A12
P3
A12
A12
P3
A12
A12
P3
A12
7'-8"
S2
58'-1 1/2"
79'-3"
#22R
4'-0"
A1
A2
A2
12'-1 1/2"
S8
A12
P2
A12
A12
P3
A12
A12
P3
A12
8"
3'-10"
3'-1 7/8"
#21R
A1
A2
A2
3'-10 7/8"
21'-1 1/2"
7'-0"
S3
A10
P1
A10
A11
P3
A11
A11
P3
A11
3'-10 7/8"
3'-1 7/8"
#20R
S1
#3a R
S8
#3a L
S1
#19R
8"
S2
#5
S2
#5
S2
#5
S2
3"
3'-1 7/8"
3'-0"
3'-1 7/8"
12'-1 1/2"
3'-1 7/8"
3'-0"
3'-1 7/8"
7'-10"
2'-0"
7'-10"
2'-0"
7'-10"
2'-0"
7'-10"
30'-9"

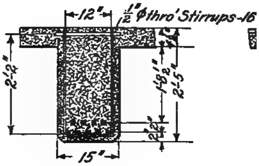

A14
A14
A14
A14
A14
A15
S7
11'-0"
71'-8"
229'-0"
P11
29'-0"
P12
8"
6'-3"
c
#9R
3'-0"
19'-9"
79'-9"
4'-0"
①
①
①
#8R
3'-0"
A5
A7
A8
A13
A13
A13
A13
A17
S3
7'-2"
P7
21'-0"
P8
P9
#7
3'-0"
20'-9"
S3
7'-2"
②
②
②
#6R
3'-0"
A2
A6
A3
8"
S2
7'-8"
A12
A12
A12
A12
A12
A12
P3
19'-4"
P4
P5
#10
2'-0"
9'-9½"
19'-8"
19'-8"
19'-7"
20'-1"
S2
7'-8"
②
②
②
#5
2'-0"
A2
A6
A3
A12
A12
A12
A12
A12
A12
S2
7'-8"
47'-8"
P3
19'-4"
P4
P5
#10
2'-0"
S2
7'-8"
②
②
②
#5
2'-0"
A2
A6
A3
A12
A12
A12
A12
A16
A16
S2
7'-8"
8"
P3
19'-4"
P4
P5
#4R
8"
4'-6"
11'-0"
8'-7"
S1
3'-0"
②
②
②
#2L
4'-4½"
A2
A6
A9
A11
A11
A18
A18
A10
S3
7'-0"
30'-9"
P3
20'-1"
P14
b
#2R
4'-4½"
8'-10"
a
S1
3'-0"
#1L
4'-6"
9"
S2
S2
S2
S2
8"
#5
#5
#5
#8R
S1
#3R
S3
#3L
S1
7'-10"
7'-10"
7'-10"
7'-10"
2'-0"
2'-0"
2'-0"
9"
4'-6"
3'-0"
4'-4½"
7'-0"
4'-4½"
3'-0"
4'-6"
3"
30'-9"



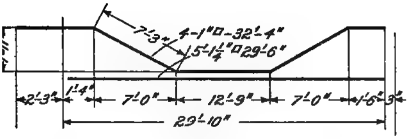
Columns ① Columns ②



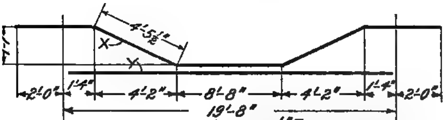
Girder A1.



Girders A4-5-7 & 8

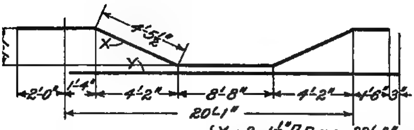


6 Girders A1



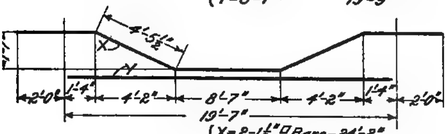
26 Girders A2 {X=2-1 1/8" □ Bars-24'-3"
Y=3-1 1/4" □ " 19'-7"

2 " A5 {X=4-1" □ " 24'-3"
Y=5-1" □ " 19'-7"



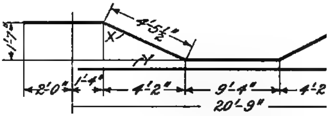
8 Girders A3 {X=2-1 1/8" □ Bars-22'-5"
Y=3-1 1/4" □ " 19'-9"

2 " A4 {X=4-1" □ " 22'-5"
Y=5-1" □ " 19'-9"



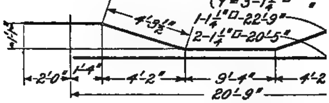
8 Girders A6 {X=2-1 1/8" □ Bars-24'-2"
Y=3-1 1/4" □ " 19'-6"

2 " A7 {X=4-1" □ " 24'-2"
Y=5-1" □ " 19'-6"

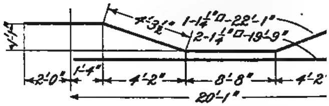


2 Girders A8 {X=4-1" □ Bars
Y=5-1" □ "

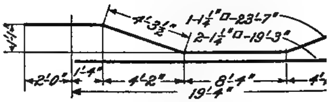
2 " A9 {X=2-1 1/8" □ "
Y=3-1 1/4" □ "



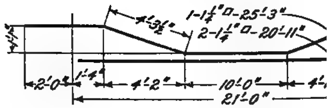
6 Beams A10



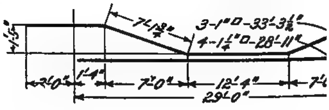
32 Beams A11



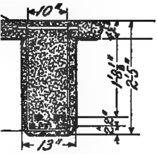
66 Beams A12



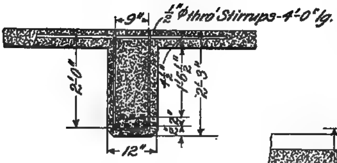
12 Beams A13



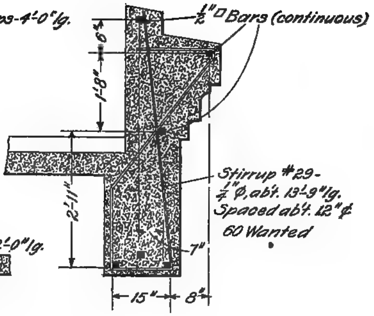
6 Beams A14



nders A2-3-6 & 9

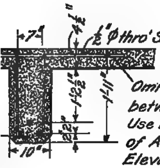


Beams A14.



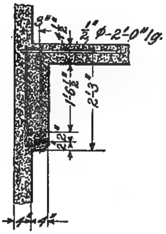
Stirrup #29-
1/4"φ, ab't. 13'-9" lg.
Spaced ab't. 12" ¢
60 Wanted

Spandrel-S8.

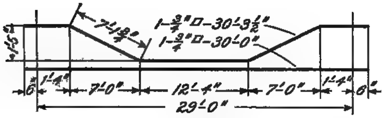


Omit these Rods in End of A10 between Elev. & Stair Openings. Use Rods 2' 0" lg. in other End of A10 & in End of A18 next to Elevator Shaft.

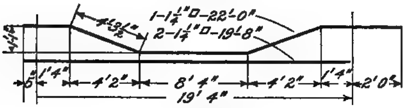
10-11-12-13-16-17 & 18.



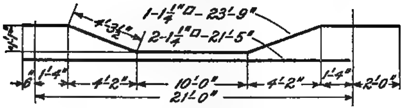
Beam A15



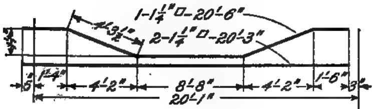
1 Beam A15



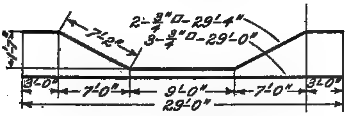
4 Beams A16



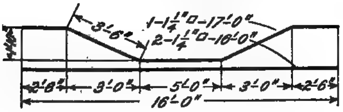
2 Beams A17



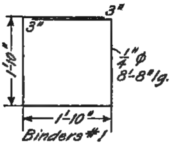
2 Beams A18



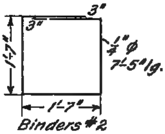
1 Spandrel S7



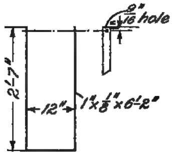
4 Spandrels S8



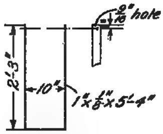
Binders #1
168 Wanted



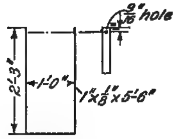
Binders #2
504 Wanted



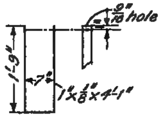
Stirrups #3
120 Wanted



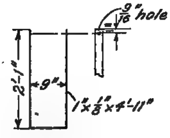
Stirrups #4
530 Wanted



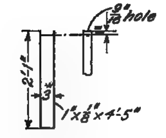
Stirrups #5
96 Wanted



Stirrups #6
1240 Wanted



Stirrups #7
96 Wanted



Stirrups #8
16 Wanted

running in both directions, following the contour. It is best in the reinforcing of domes to use light rods spaced close, rather than heavy rods, because the lighter rods can be kept in position better while the concrete is being placed; also, they tend to strengthen the dome more uniformly throughout and provide greater elasticity to the construction, which is desirable in such structural features as domes and vaults.

FACTORY CONSTRUCTION

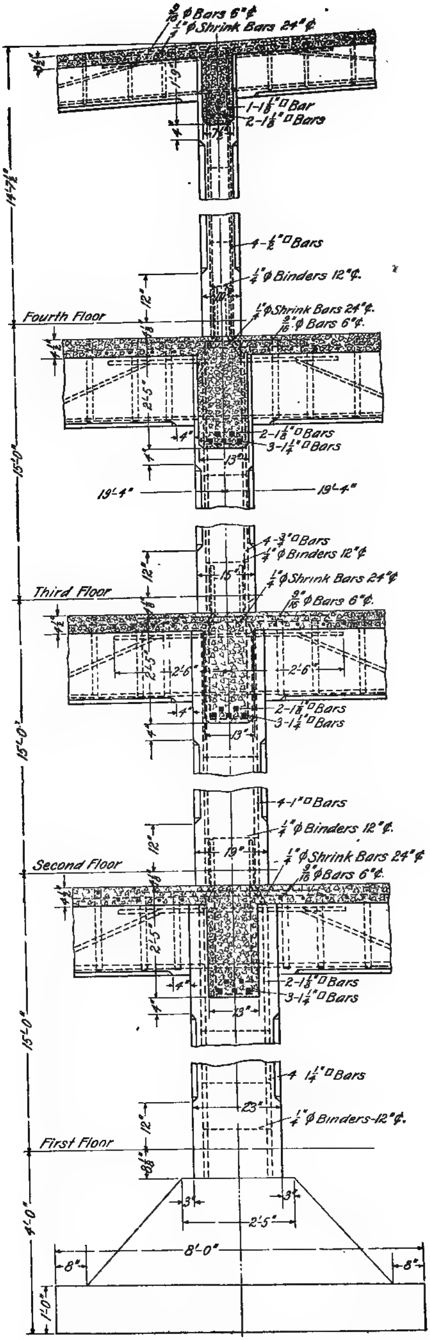
38. Reinforced concrete has its greatest use in factory construction, as by this means a fireproof building may be erected at a reasonable cost. Fig. 36 shows a typical construction plan of a reinforced-concrete floor for a large four-story factory building. This plan shows the definite location of the beams, girders, columns, and piers, each beam, girder, and column being indicated by a specific mark. It also shows the direction of the slab reinforcement, and likewise the shrinkage rods.

Fig. 37 shows the detail sheet that accompanies the floor plan. This sheet contains the detailed sections through some of the spandrels, band courses, and cornice, as well as the working drawings for the reinforcement for the beams and girders. The latter drawings, which are skeleton diagrams, show the length of the several rods and the way in which they are bent to form the camber, or truss, rods.

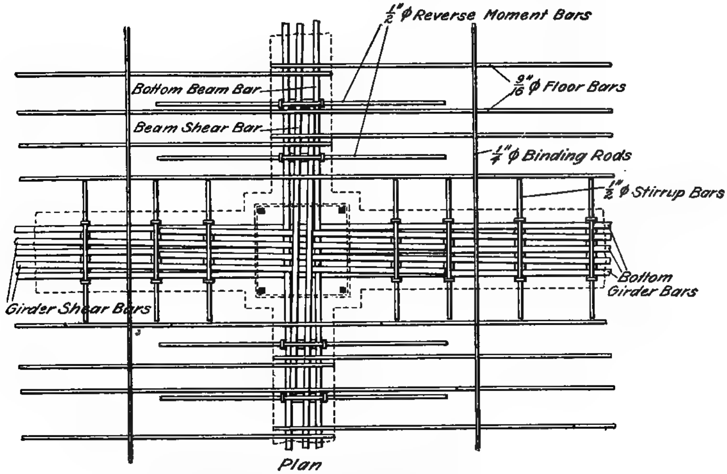
In addition to the plans just mentioned, it is customary to make a large detail sheet of the column and girder connections, as illustrated in Fig. 38. In this figure are given the story heights, the method of arranging the beam, girder, and column connections at the several floors, and the splice connections of the column rods.

In order to show the manner in which the several rods of the beam and girder connections lap and intertwine at the column supports, the detail drawing, Fig. 39, is given. This figure also shows the shrinkage or binding rods.

The slab reinforcement of this construction is shown in the typical panel sketch, Fig. 40. The reinforcing rods of the



Section through Girders Showing Columns ②


$\frac{1}{2}$" ⌀ Reverse Moment Bars
Bottom Beam Bar
$\frac{9}{16}$" ⌀ Floor Bars
Beam Shear Bar
$\frac{1}{4}$" ⌀ Binding Rods
$\frac{1}{2}$" ⌀ Stirrup Bars
Bottom Girder Bars
Girder Shear Bars

Plan
Showing Typical Arrangement of
Steel Reinforcement in Girders,
Beams and Columns at Bearing on Latter.

FIG. 39

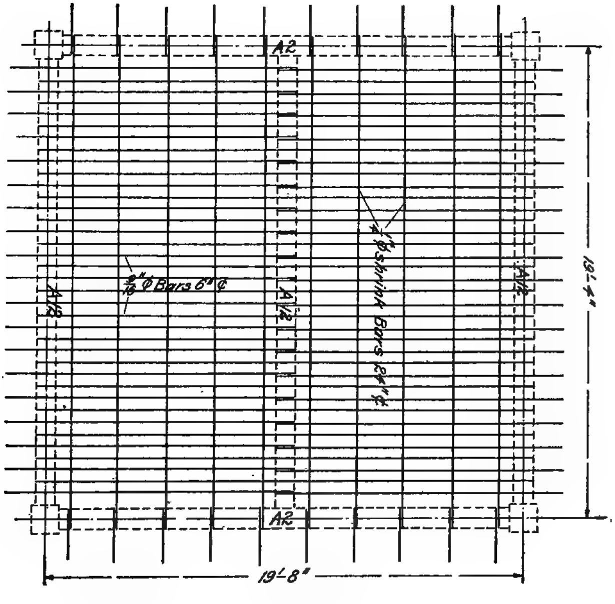

A2
$\frac{9}{16}$" ⌀ Bars 6" ℄
A1/2
A1/2
A1/2
$\frac{1}{4}$" ⌀ Shrink Bars 24" ℄
19'-4"
A2
19'-8"

FIG. 40

slabs are $\frac{9}{16}$-inch round bars spaced 6 inches from center to center, while the shrinkage rods are $\frac{1}{4}$-inch round bars placed 24 inches on centers.

The construction is based on the engineering practice of the American Concrete Steel Company. As there is much to be learned from the careful method of designing illustrated here, the various views will be taken up and explained in detail as far as required.

39. Small reference letters have been added to the drawing shown in Fig. 36 in order to indicate some of the parts referred to in the description. In a regular construction drawing, these letters are omitted. The other letters and marks are found in most drawings of this class. Marks enclosed in circles, such as (*P2*), indicate the different kinds of floor panels. Those having similar marks are alike; that is, they are of the same length, width, and cross-section, and have an equal number of reinforcing bars. The girders are indicated by marks *A1*, *A2*,. . . to *A9*, inclusive, and beams by *A10*, *A11*,. . . to *A18*, inclusive. Piers are indicated by single numbers, from *1* to *22*, inclusive, and the spandrels between the piers are marked with the letter *S*, to which are added the numerals from *1* up to and including *8*. There are two varieties of columns indicated by the marks ① and ②.

In the floor plan of the first floor, Fig. 36, only one-half of the floor is shown, as both halves are arranged symmetrically about the center line. The opening *a* is used for an elevator, while openings *b* and *c* may be used for stairs and a large elevator.

40. Detail drawings are supposed to be made of all the members, but some of them are omitted in the illustrations; those shown, however, are sufficient to illustrate the method of applying the dimensions. The detail dimensions of the reinforcements are here given by means of a series of diagrams in Fig. 37. At the middle of the left-hand side of the illustration will be found the dimensions of the reinforcements for girders *A1*, there being six of these for the whole floor. It will be noticed that the left horizontal end of the truss bar is longer than the right one. The reason for this will be evident on consulting Fig. 36, in which it is shown that one end of the girder is supported by a pier and the other by a column. The end of the truss rod resting in the column is longer, because it is to extend beyond the column into the adjoining girder. In the diagrams of the beam reinforcements also, it will be seen that in some cases there is some difference in length between the ends of the truss rods. For instance, beam *A10* requires different lengths of the ends, because one end has to rest in the pier *# 3* and the other in the column ②. Under this diagram, between two vertical center lines, is placed the dimension *20′-9″*. The left-hand center line is that of the column, the right-hand one that of the pier. The dimension *3″* indicates the distance between the center line of the pier and the ends of the reinforcing rods. The number of sets of reinforcing rods required for each kind of beams is also indicated under each diagram. For instance, the note, *66 Beams A12*, shows that sixty-six sets of reinforcing rods are required of the kind marked *A12*.

41. In the elevation of column ②, Fig. 38, the dimension *15′* indicates the distance between the finished surfaces of the first and second floor. The binders surrounding the reinforcing rods, as shown, are to be made of ¼-inch round wire spaced at distances of 1 foot, center to center. The remaining parts on the drawing are represented in the usual manner and require no explanation. The mark □ is used to indicate the word *square* and the mark Φ, to indicate the word *round*.

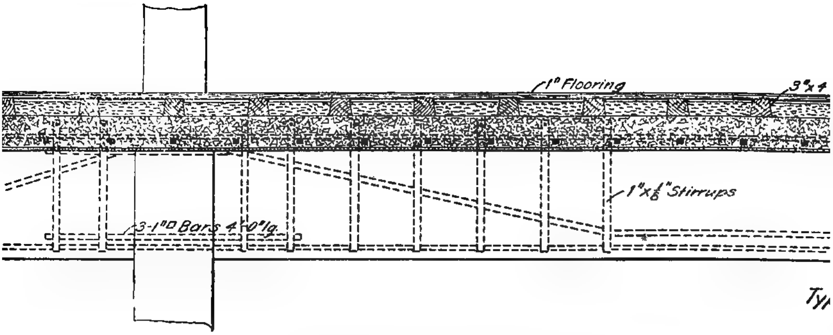
STORE AND LIGHT OFFICE-BUILDING CONSTRUCTION

42. Reinforced concrete is also used extensively for light office-building and store construction. Typical details for a three-story store and office building of this character are shown in Fig. 41. This drawing illustrates in particular the use of long-span floor slabs supported upon beams running across the building. There are no girders connecting the columns; consequently, the beams run directly into the columns. This construction is particularly applicable to buildings of this type, provided the floor loads are light and a fireproof building of low cost is required. The illustration shows a typical elevation of a beam, and a section indicating the story heights and details of the beam and column connections. A typical pilaster, or exterior pier, is also shown.

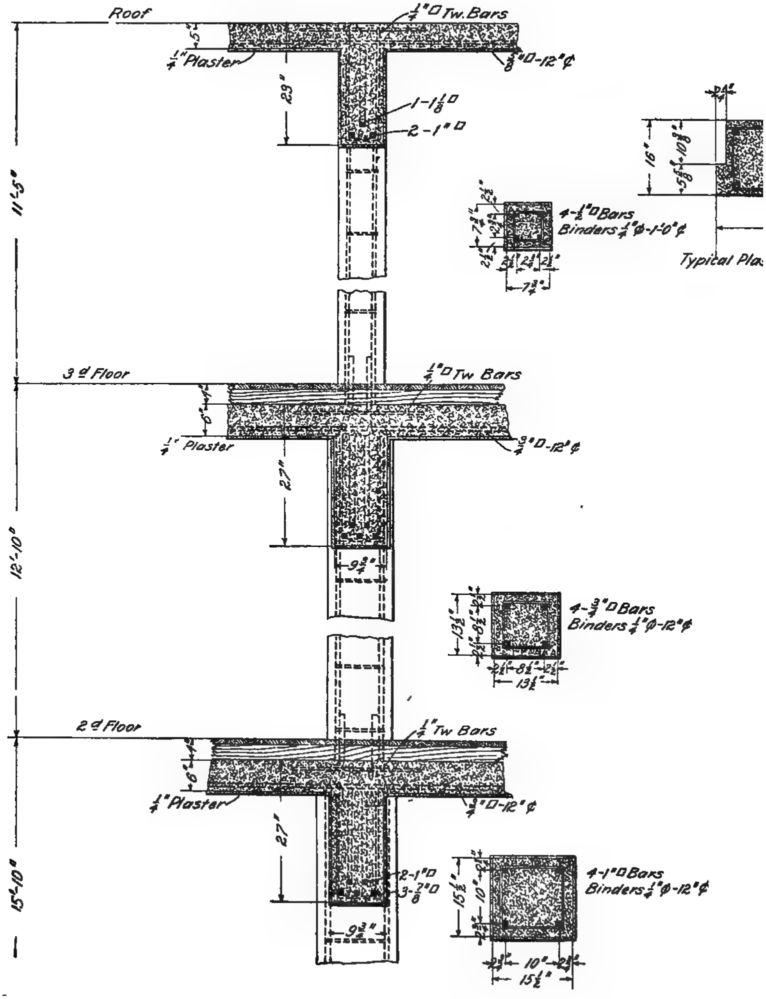
43. The plan of the building, Fig. 42, shows the position of the wall piers and columns, together with the spacing of the beams. The plan likewise shows the solid concrete wall on one side of the building, and a note adjacent thereto gives in detail the reinforcement for this wall.

In Fig. 43 are shown several detail sections through spandrels and string-courses. By referring to the plan, the position of these several sections will be seen. In Fig. 44 the details for bending the reinforcement are given, the marks for the several pieces referring to similar marks on the plan.

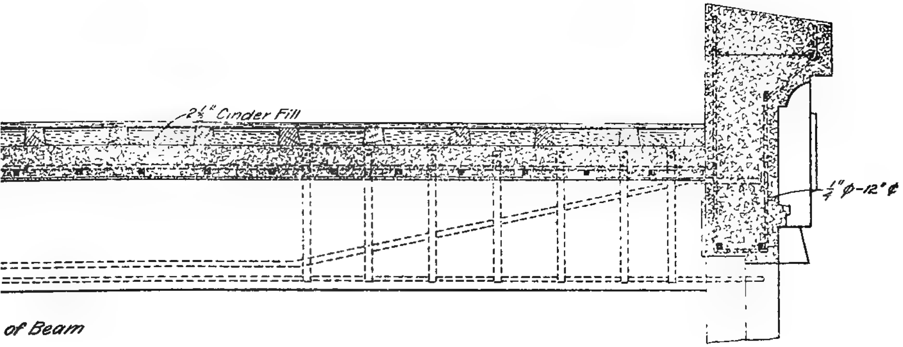
44. Some further explanation is needed in order that these drawings just mentioned may be fully understood. In the plan, Fig. 42, distinction is made between beams and partition walls by indicating the former by means of dotted lines. For instance, the elevator shaft *a*, in view (*a*), has walls on the outside and on one other side, these walls being shown in section. At the other two sides, the shaft is surrounded by beams shown by dotted lines. It is also shown that panel (*P5*) is resting on three partition walls, one of which, at *b*, is joined to beam *#1*. At other places, two parallel

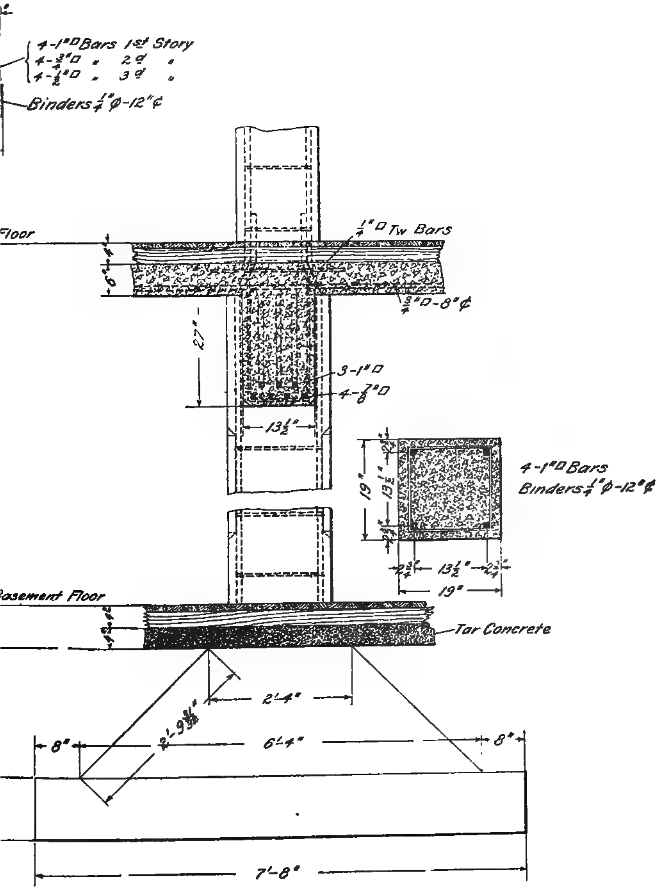


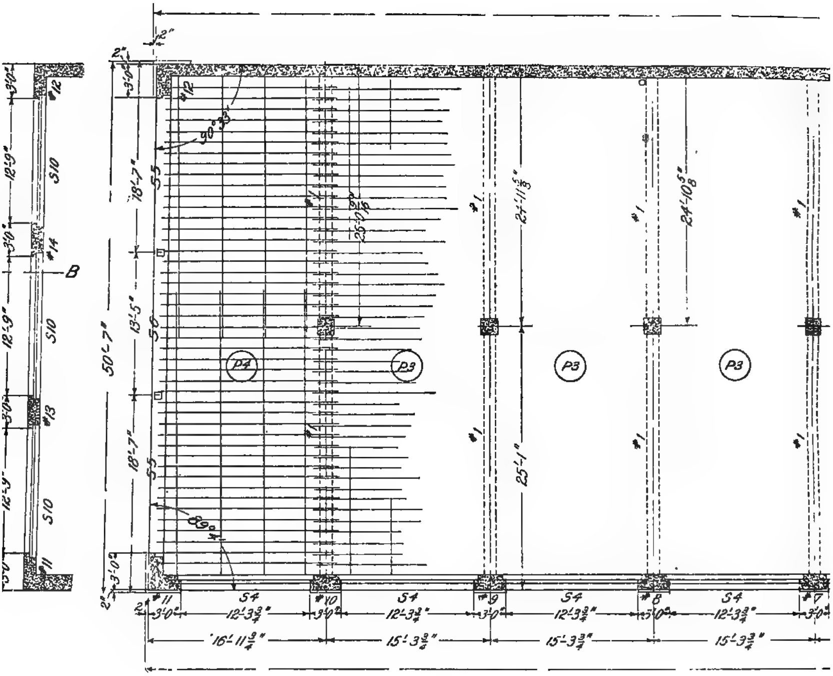
1" Flooring
3"x4
1"x⅛" Stirrups
3-1"□ Bars 4'-0" lg.
Ty

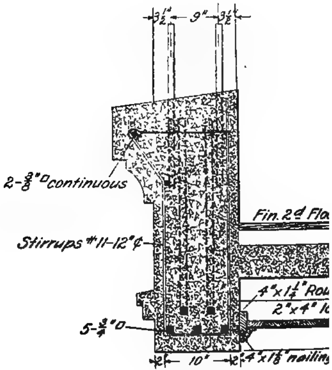


Roof
5"
¼" Plaster
¼"□ Tw. Bars
⅜"□-12"¢
23"
1-1⅛□
2-1"□
11'-5"
4-½"□ Bars
Binders ¼"φ-1'-0"¢
7¾"
2½"
2¼"
7¾"
16"
10⅜"
5⅝"
Typical Pla
3d Floor
6"
¼" Plaster
¼"□ Tw Bars
¾"□-12"¢
27"
9¾"
12'-10"
4-¾"□ Bars
Binders ¼"φ-12"¢
13½"
8½"
2½"
2d Floor
6"
¼" Plaster
¼" Tw Bars
¾"□-12"¢
27"
2-1"□
3-⅞"□
9¾"
15'-10"
4-1"□ Bars
Binders ¼"φ-12"¢
15½"
10"
2¾"

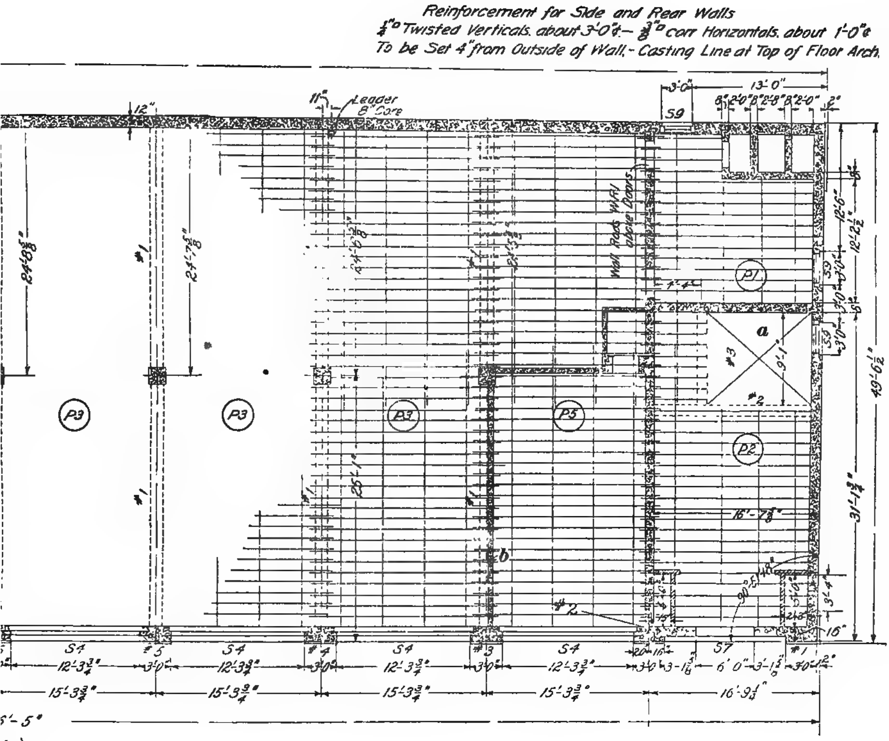

2½" Cinder Fill
¼"ϕ-12"¢
of Beam


4-1"□ Bars 1st Story
4-¾"□ " 2d "
4-½"□ " 3d "
Binders ¼"ϕ-12"¢
Floor
¼"□ Tw Bars
6"
⅜"□-8"¢
27"
3-1"□
4-⅞"□
13½"
4-1"□ Bars
Binders ¼"ϕ-12"¢
19"
13½"
2¾"
2¾"
13½"
2¾"
19"
Basement Floor
Tar Concrete
2'-4"
2'-9 31/32"
8"
6'-4"
8"
7'-8"


B
50'-7"
18'-7"
13'-5"
3'-0"
12'-9"
S10
#11
#12
#13
#14
S5
S6
90°33'
89°4'
25'-0 5/16"
24'-11 5/8"
24'-10 5/8"
25'-1"
#1
P4
P3
S4
#7
#8
#9
#10
12'-3 3/4"
16'-11 3/4"
15'-3 3/4"
2"

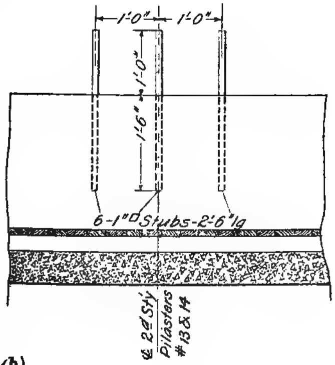

3 1/2"
9"
2-3/8" ▫ continuous
Stirrups #11-12" ¢
5-3/4" ▫
10"
2"
Fin. 2d Flo
4"×1 1/4" Rou
2"×4" lo
4"×1 3/8" nailing

SECTION B-B

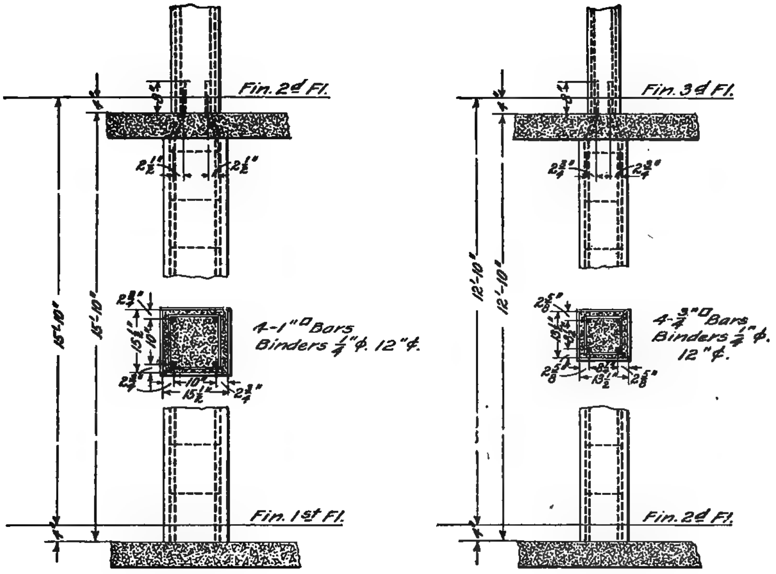


Reinforcement for Side and Rear Walls
¼"□ Twisted Verticals, about 3'-0"¢ – ⅜"□ corr Horizontals, about 1'-0"¢
To be Set 4" from Outside of Wall, – Casting Line at Top of Floor Arch.

(a)

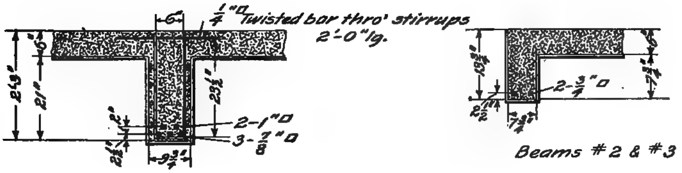


(b)



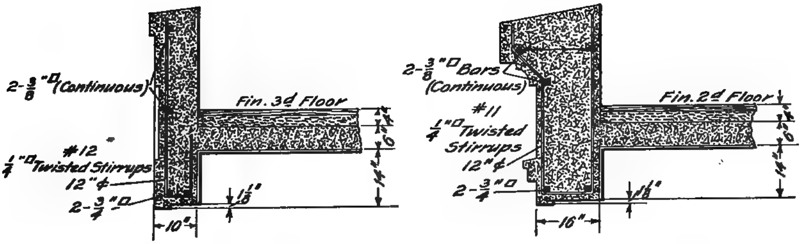
1st Story Cols.

2d Story Cols.



Beams #1.

Beams #2 & #3



Spandrels at 3d Fl. Level.

Spandrels at 2d Fl. Level.

FIG. 43

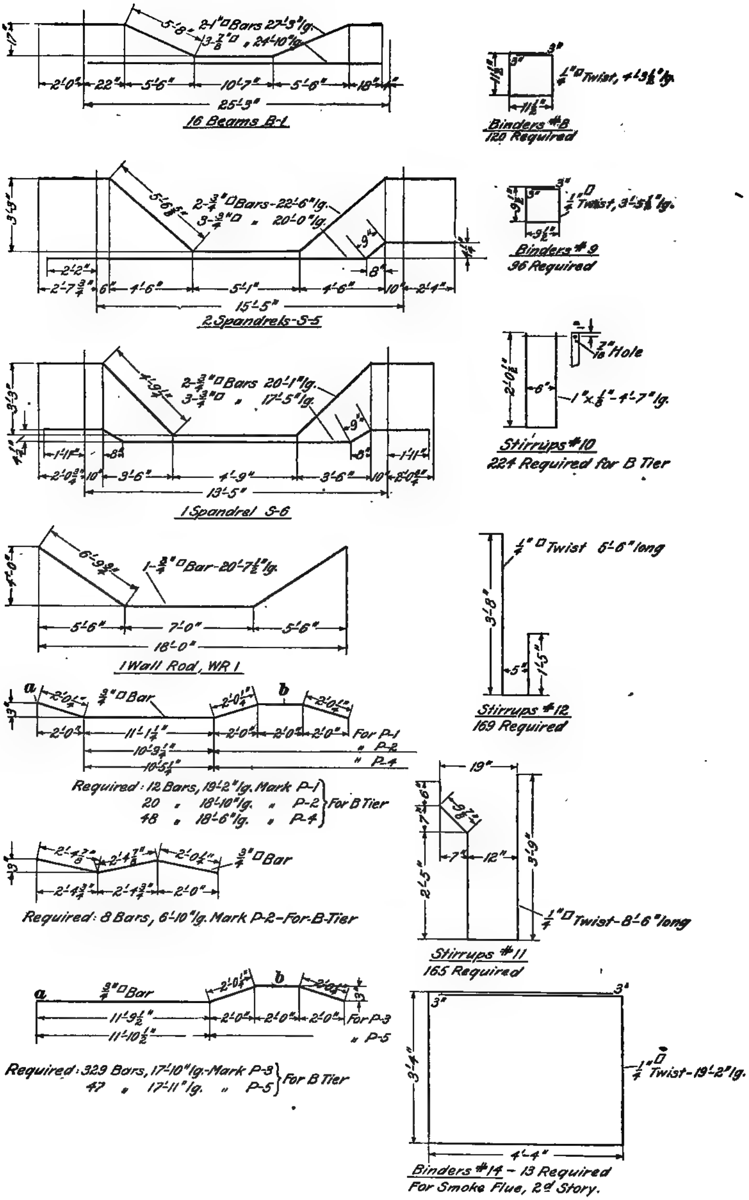

1'-7"
2-1"□Bars 27'-3"lg.
3-7/8"□ " 24'-10"lg.
5'-8"
2'-0" 22" 5'-6" 10'-7" 5'-6" 18" 1"
25'-3"
16 Beams B-1
3" 3" 11½" 11½"
¼"□Twist, 4'-3½"lg.
Binders #8
120 Required
3'-5"
5'-6⅞"
2-¾"□Bars-22'-6"lg.
3-¾"□ " 20'-0"lg.
9" 1'-4"
2'-2" 8"
2'-7¾" 6" 4'-6" 5'-1" 4'-6" 10" 2'-4"
15'-5"
2 Spandrels-S-5
3" 3" 9½" 9½"
¼"□Twist, 3'-5½"lg.
Binders #9
96 Required
3'-3"
4'-9¼"
2-¾"□Bars 20'-1"lg.
3-¾"□ " 17'-5"lg.
9" 4¼"
1'-11" 8" 8" 1'-11"
2'-0¾" 10" 3'-6" 4'-9" 3'-6" 10" 2'-0¾"
13'-5"
1 Spandrel S-6
2'-0½" 6"
7/16"Hole
1"x⅛"-4'-7"lg.
Stirrups #10
224 Required for B Tier
4'-0"
6'-9¾"
1-¾"□Bar-20'-7½"lg.
5'-6" 7'-0" 5'-6"
18'-0"
1 Wall Rod, WR 1
¼"□Twist 5'-6"long
3'-8" 5" 1'-5"
Stirrups #12
169 Required
a b
2'-0¼" ¾"□Bar 2'-0¼" 2'-0¼"
3"
2'-0" 11'-1½" 2'-0" 2'-0" 2'-0" For P-1
10'-9¾" " P-2
10'-5¼" " P-4
Required: 12 Bars, 19'-2"lg. Mark P-1
20 " 18'-10"lg. " P-2
48 " 18'-6"lg. " P-4
For B Tier
2'-4⅞" 2'-4⅞" 2'-0¼" ¾"□Bar
3"
2'-4¾" 2'-4¾" 2'-0"
Required: 8 Bars, 6'-10"lg. Mark P-2-For-B-Tier
19" 6" 7½" 9⅞"
7" 12"
2'-5" 5'-9"
¼"□Twist-8'-6"long
Stirrups #11
165 Required
a b
¾"□Bar 2'-0¼" 2'-0¼" 3"
11'-9½" 2'-0" 2'-0" 2'-0" For P-3
11'-10½" " P-5
Required: 329 Bars, 17'-10"lg.-Mark P-3
47 " 17'-11"lg. " P-5
For B Tier
3" 3"
3'-4"
¼"□Twist-19'-2"lg.
4'-4"
Binders #14 – 13 Required
For Smoke Flue, 2d Story.

FIG. 44

dotted lines indicate that the second floor is resting on beams marked *# 1*.

45. Plans used in architectural drawing do not follow the same rules as those applying to mechanical drawing. In the plan, Fig. 42 (*a*), some advantage is gained by combining several plans into one. It will be noticed that in addition to the columns, pilasters, and walls of the first story, the second-floor panels with their reinforcement and the supporting beams are shown. A sectional plan of the second-floor front is shown at the left of the other plan, because at this point the two plans differ, the first story having cast-iron columns only, while the second floor has piers with windows intervening. A section through a spandrel of the second-floor front is shown in Fig. 42 (*b*).

46. Distinction is made in Fig. 44 between the beams for the various floors by adding to the number of the beam the letters *A*, *B*, and *C*, which indicate the first, second, and third floor, respectively. The floors are in general referred to as *A Tier*, *B Tier*, and *C Tier*. In this drawing, details are given for the *B Tier* only. The expression *16 Beams B–1*, found in Fig. 44, means that sixteen sets of rods are required for sixteen beams marked *# 1* in *Tier B*, or the second floor.

In the details for the reinforcing rods of the floor slabs, in Fig. 44, only those going in the *B Tier* are referred to. It will be seen that the total length of the bent bars, as found from the diagram, does not agree with the length given in the table below the figure. The length of a bar for a *P–1* slab is, in the diagram, 19 feet 1¼ inches, but in the table it is 19 feet 2 inches. The reason for this difference is that the table gives the length of the straight bar, while the diagram gives the length of a bent bar. There is some difference in form between the rods intended for the slabs *P–1* and those used in slabs *P–3*. The former have one end *a* resting in the wall and the other end *b* resting on a beam. In the rods for slab *P 3*, both ends rest on beams, and one end is made straight and the other is bent up as shown. The bars overlap the beams by equal amounts at both ends, which causes the bent part of the bar to come a little off the center line of the beam. The bar is bent up so that some metal can be put in the top of the slab to take up the reverse bending moment that occurs there. The bars are so placed that the end of every other bar that passes over one beam is bent; that is, in placing the bars, first one bar is laid with the bent end pointing one way and then the adjacent bar is placed with the bent end pointing the other way, and so on.

The word *mark* used in Fig. 44, as *Mark P–1*, indicates that the sign *P–1* is to be painted or tagged on that piece of reinforcement in the shop, so that when it comes to the building, it can be readily distinguished and put in its proper place.

CONSTRUCTION WITH TILE FLOORS AND KAHN BARS

47. In Fig. 45 is shown the framing plan of a large factory building to be constructed with hollow tile floors and concrete joists and girders. The concrete work is to be reinforced throughout with Kahn truss bars. The markings upon the plan are typical of a reinforced-concrete framing plan, and refer to the details, bill of material, and schedule. The bill of material required for the plan and the schedule giving the detail of the sizes of the reinforcement are given in tables shown with this figure.

The construction of the floor system is shown in Fig. 46. In this figure, the details of the floor and girder construction, together with its reinforcement, are clearly set forth. The lintel construction is also illustrated in the sections on *d–d*, *e–e*, and *k–k*. The footings for this building, shown in Figs. 47 and 48, are also reinforced concrete, and are of special interest on account of their irregular shape and because there are a number of examples of two-column footings. Their position may be observed from the plan, following the column numbers.

The stair construction, shown in Figs. 49 and 50, is also interesting. These figures illustrate the adaptability of concrete stairs to irregular spaces and show how they may be designed to meet the peculiar conditions that arise in every-day practice.

It is recommended that a careful study be made of these plans and details, because they illustrate the best current practice in reinforced-concrete design and also give an excellent idea of the method of preparing the drawings of this kind of work for field construction.

48. As these drawings and schedules have reference to reinforced-concrete construction only, no indication is given of the fact that brick is to be used for the walls and spandrels. Some of the places where the brick construction adjoins the reinforced concrete will be pointed out in the subsequent description.

Referring to the drawings in detail, it will be seen from the plan, Fig. 45, that the columns are indicated by numbers and the beams by capital letters. In this plan, the beams are indicated by dotted lines and the reinforcing Kahn bars in the floor slabs by full thin lines. Additional information about the tiles, reinforcing bars, and anchors is also given in the plan. Thus, on the slab supported by beams *M*, *N*, *E*, and *S* is found the note *7–½″ × 1½″ Anchors, Mk. 64*, which means that seven Kahn bars ½ in. × 1½ in., marked *64*, are to act as anchors. In the *Bill of Material* it is found that these are 4 feet 6 inches long, and that the total number required is eighty-six. They are used in an inverted position; that is, with the prongs turned down.

The note, *10″ Tile + 1″ Concrete, 13–¾″ × 2″ K. Bars Mk. 106, 16″ C. to C.*, means that the thickness of the floor tiles is 10 inches, on top of which is a layer of concrete 1 inch thick; also, that there are thirteen reinforcing Kahn bars, ¾ in. × 2 in., marked *106*, and that they are spaced 16 inches center to center. From the bill of materials it is found that the bars are 14 feet 9 inches long. The general arrangement of the tiles, the intervening joists of Kahn bars, and the concrete layer may be seen in the section on *a–a*, Fig. 46. This section also shows that the joists between columns are double, as at column *#31*. The remaining part of the plan requires no further explanation.

Some of the abbreviations and notes found in the *First-Floor Beam Schedule*, Fig. 45, require further explanation. For instance, referring to the beam marked *L*, the abbreviation *Std.* for *standard* is found in the seventh column. This means that the Kahn bar is made with a clear section without any stirrups at its center, according to the standard, as shown in Fig. 51. The abbreviation *Ctr.* for *center*, found in the same column, indicates that the central part of the Kahn bar is also sheared and bent up into stirrups, as shown in Fig. 52. The word *Cup*, found in the fifth column, indicates that the reinforcing bar in question is a *cup bar*. This bar has been described in a previous Section. The note *14″ Tee each side*, found in the first line, indicates that the reinforced beam has a **T**-shaped extension at its upper side, formed by incorporating a part of the floor slab at either side into the beam, as shown in Fig. 53. The width of beam, inclusive of the extensions, is, in this case, to be 14 inches.

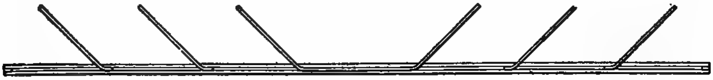


FIG. 51

49. In the section on *a–a*, Fig. 46, the lintel, or beam, *I*, between columns *#43* and *#44*, Fig. 45, is shown in cross-section. The under side of the lintel is rabbeted for the window frames. As its upper side is only 12 inches above the floor level, the spandrel must be laid in brick to the required height for the window sill. In the section on *b–b* are shown the outside of the lintel *I* and columns *#45* and *#46*, together with the method of reinforcing and joining the lintel to the columns.

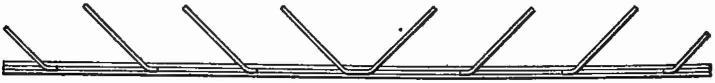
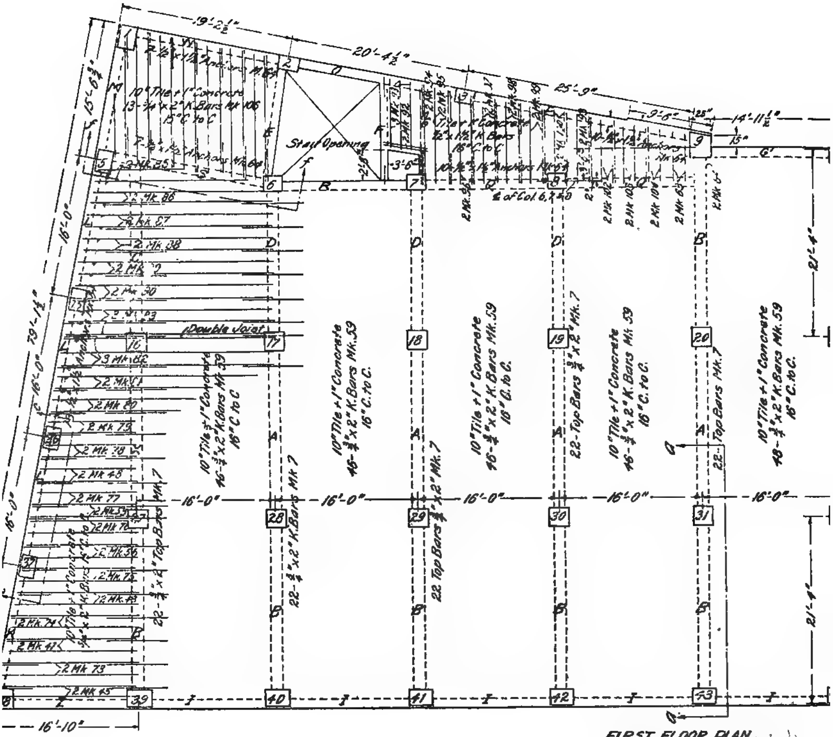


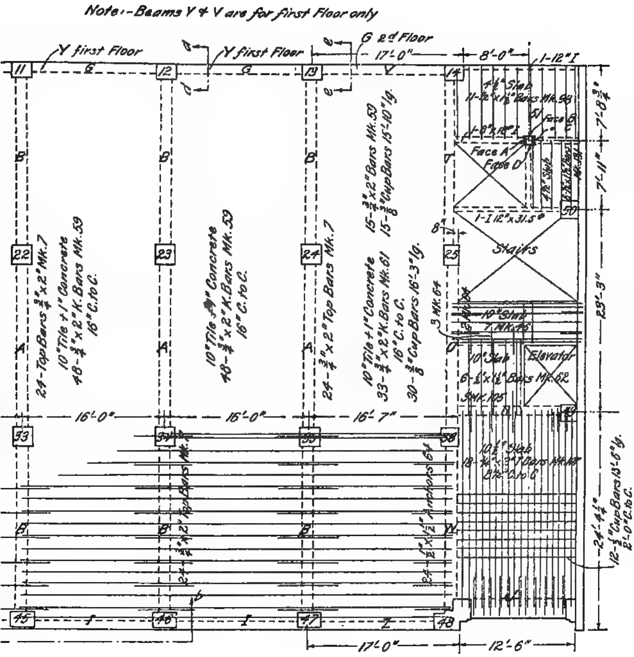
FIG. 52



FIRST FLOOR PLAN
Scale 1/8" = 1'-0"
Second Floor Similar.

Beam		Bars in one Beam						Remarks
Size	No	No	Size	Length	Shear	10th Diag'l	Mk	
		3	1¾"x2¾"	20'-3"	Std	30	39	14" Tee each side
16x22	8	2	1¼" Cup	32'-0"				
		[illegible]	1¾"x2¾"	20'-3"	Std.	30	39	12" Tee each side
16"x22"	1	1	1"x3"	16'-0"	Ctr.	24	34	
		2	1⅛" Cup.	32'-0"				
		3	1¾"x2¾"	21'-0"	Std	30	40	14" Tee each side.
16"x22"	14	2	1¼" Cup.	30'-0"				
	1	2	1¾"x2¾"	21'-0"	Std	30	40	12" Tee each side
16"x22"		1	1"x3"	15'-0"	Ctr	24	41	
		2	1" Cup	30'-0"				
		2	1¾"x2¾"	13'-6"	Std	30	1	12" Tee each side
16"x22"	3	1	1"x3"	13'-6"	Ctr	24	42	
		2	1" Cup	26'-0"				
10"x16"	1	2	¾"x2"	14'-0"	Std.	12	43	
10"x16"	1	2	¾"x2"	12'-9"	Std	12	44	
12"x29¾"	2	2	¾"x2"	15'-11"	Std.	12	45	
		1	⅝" Cup.	25'-0"				

Y	12"x20"	2	2	¾"x2"	15'-11"	Std.	[illegible]
			1	⅝" Cup.	25'-0"		
I	15"x29¾"	8	2	¾"x2"	15'-11"	Std.	12
			1	⅝" Cup.	25'-0"		
Z	10½"x29¾"	2	2	¾"x2"	15'-11"	Std.	12
			1	⅝" Cup.	25'-0"		
K	10½"x29¾"	1	2	¾"x2"	15'-2"	Std.	12
			1	¾"x2"	11'-0"	Ctr.	12
			1	⅝" Cup.	20'-0"		
L	15"x29¾"	3	2	¾"x2"	15'-9"	Std	12
			1	⅝ Cup.	25'-0"		
M	15"x13½"	1	2	¾"x2"	15'-0"	Std.	12
			1	⅝" Cup.	20'-0"		
N	12"x24"	1	2	1¾"x2¾"	18'-9"	Std.	30
			1	¾" Cup.	27'-0"		
			2	1"x3"	20'-2"	Std.	24
O	12"x24"	1	2	¾" Cup.	32'-0"		
			2	1"x3"	15'-0"	Ctr.	24
P	14"x28"	1	2	1¾"x2¾"	27'-3"	Std.	[illegible]
			2	¾" Cup.	35'-0"		
Q	10"x20"	2	2	¾"x2"	15'x11"	Std.	12
			1	⅝" Cup.	25'-0"		



R	10"x20"	1	2	1"x3"	15'-11"	Std	24	53	12" Tee one side
			1	⅝" Cup. B	25'-0"				
S	16"x24"	1	2	1¾"x2¾"	20'-0"	Std	30	54	8" Tee each side
			2	1⅛" Cup.	28'-0"				
			1	1¾"x2¾"	20'-0"	Ctr	30	55	
T	12"x22"	1	2	1¾"x2¾"	21'-0"	Std	30	40	16" Tee one side
			1	1¼" Cup. B	30'-0"				
			1	1"x3"	15'-0"	Ctr	24	41	
U	16"x22"	1	2	1¾"x2¾"	20'-0"	Std	30	54	14" Tee one side
			1	1¼" Cup. B.	32'-0"				
			1	1"x3"	15'-0"	Ctr.	12	41	
V	12"x20"	1	2	¾"x2"	15'-11"	Std	12	45	
			1	⅝" Cup	22'-6"				
W	16"x22"	1	2	1¾"x2¾"	21'-0"	Std	30	40	14" Tee one side
			1	1¼" Cup B.	30'-0"				
			1	1"x3"	15'-0"	Ctr.	12	41	
			2	¾"x2"	13'-0"	Std	12	56	
H	14"x18"	1	1	¾"x2"	9'-6"	Ctr	12	57	
			2	⅝" Cup.	16'-0"				
			2	¾"x2"	12'-0"	Std.	12	58	
J	28"x27¼"	1	1	¾"x2"	9'-6"	Ctr.	12	57	
			2	⅝" Cup	16'-0"				

FIRST FLOOR
Bill of Material

Kahn Bars

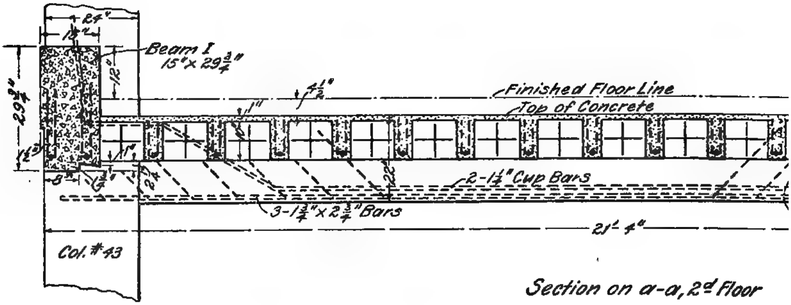
No	Size	Length	Shear		Mk.
6	1¾"x2¾"	18'-6"	Std	30	1
26	" "	20'-3"	"	"	39
48	" "	21'-0"	"	"	40
2	" "	18'-9"	"	"	50
2	" "	27'-3"	"	30	52
4	" "	20'-0"	"	"	54
1	" "	20'-0"	Ctr	30	55
1	1"x3"	16'-0"	Ctr.	24	34
6	" "	13'-0"	"	"	41
3	" "	13'-6"	"	"	42
2	" "	20'-2"	Std.	"	51
2	" "	15'-11"	"	"	53
206	¾"x2"	6'-6"	Ctr.	12	7
4	" "	14'-0"	Std.	"	43
2	" "	12'-9"	"	"	44
36	" "	15'-11"	"	"	45
7	" "	13'-9"	"	"	46
6	" "	15'-0"	"	"	47
3	" "	11'-0"	Ctr.	"	48
6	" "	15'-9"	Std.	"	49
5	" "	13'-0"	"	"	56
2	" "	9'-6"	Ctr	"	57
4	" "	12'-0"	Std.	"	58
391	" "	15'-10"	"	"	59
33	" "	16'-3"	"	"	61
2	" "	15'-6"	"	"	73
2	" "	14'-6"	"	"	74
3	" "	13'-6"	"	"	75
2	" "	12'-5"	"	"	76
2	" "	11'-6"	"	"	77
2	" "	10'-6"	"	"	78
2	" "	10'-0"	"	"	79
[illegible]	" "	9'-6"	"	"	80
2	" "	9'-0"	"	"	81
3	" "	8'-6"	"	"	82
4	" "	8'-0"	"	"	83
2	" "	5'-0"	"	"	85
2	" "	5'-6"	"	"	86
3	" "	6'-0"	"	"	87
2	" "	6'-6"	"	"	88
2	" "	7'-0"	"	"	89
2	" "	7'-6"	"	"	90
13	" "	14'-9"	"	"	106
18	" "	23'-0"	"	"	107
6	½"x1½"	8'-9"	Std.	6"	62
14	" "	6'-0"	"	"	63
86	" "	4'-6"	Anchors		64
2	" "	5'-0"	Std.	6"	92
16	" "	8'-0"	"	"	93
2	" "	11'-6"	"	"	94
2	" "	11'-0"	"	"	95
2	" "	10'-6"	"	"	96
2	" "	10'-0"	"	"	97
2	" "	9'-6"	"	"	98
2	" "	9'-0"	"	"	[illegible]
2	" "	8'-6"	"	"	100
2	" "	7'-6"	"	"	102
2	" "	7'-0"	"	"	103
2	" "	6'-6"	"	"	104
3	" "	5'-0"	Ctr.	"	105
6	" "	7'-8"	Std.	"	131

Cup Bars

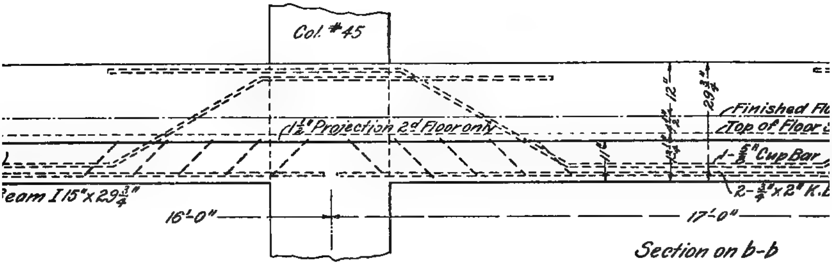
No.	Size	Length
17	1¼"	32'-0"
30	"	30'-0"
2	1⅛"	32'-0"
2	"	28'-0"
2	1"	30'-0"
6	1"	26'-0"
1	¾"	27'-0"
2	"	32'-0"
2	"	35'-0"
2	⅝"	20'-0"
1	"	22'-6"
20	"	25'-0"

Cup Bars

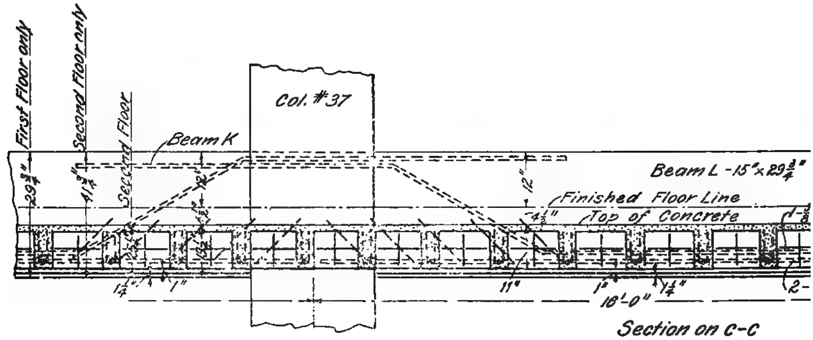
No.	Size	Length
4	⅝"	16'-0"
30	⅜"	16'-3"
15	"	15'-10"
12	½"	13'-6"



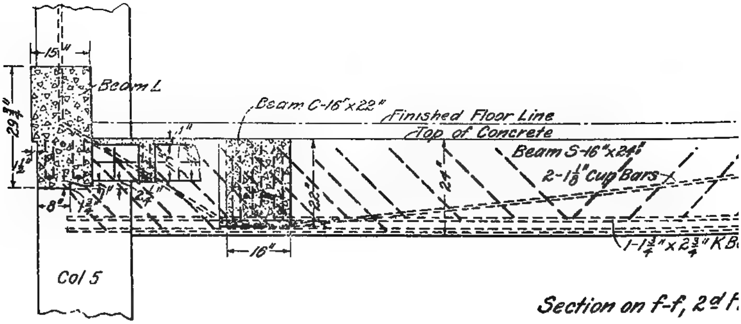
Section on a-a, 2d Floor



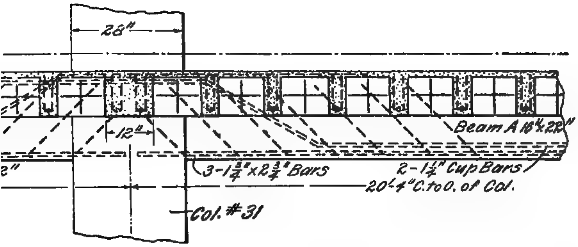
Section on b-b

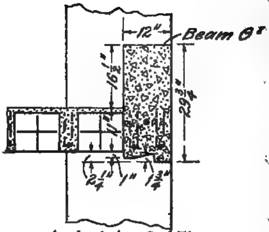


Section on c-c

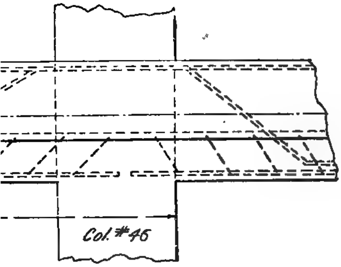


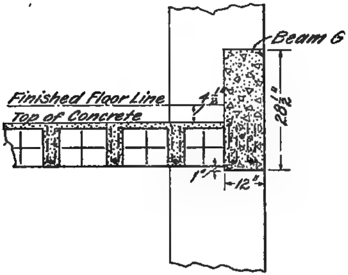
Section on f-f, 2d F



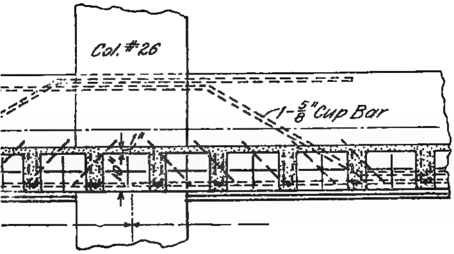


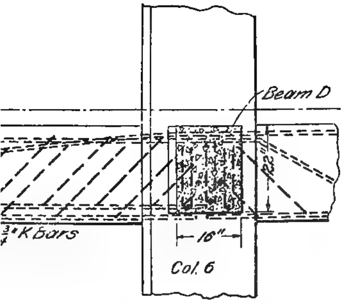
Section on k-k, 1st & 2d Floors

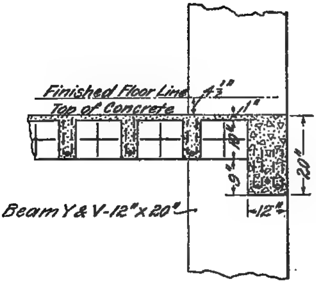




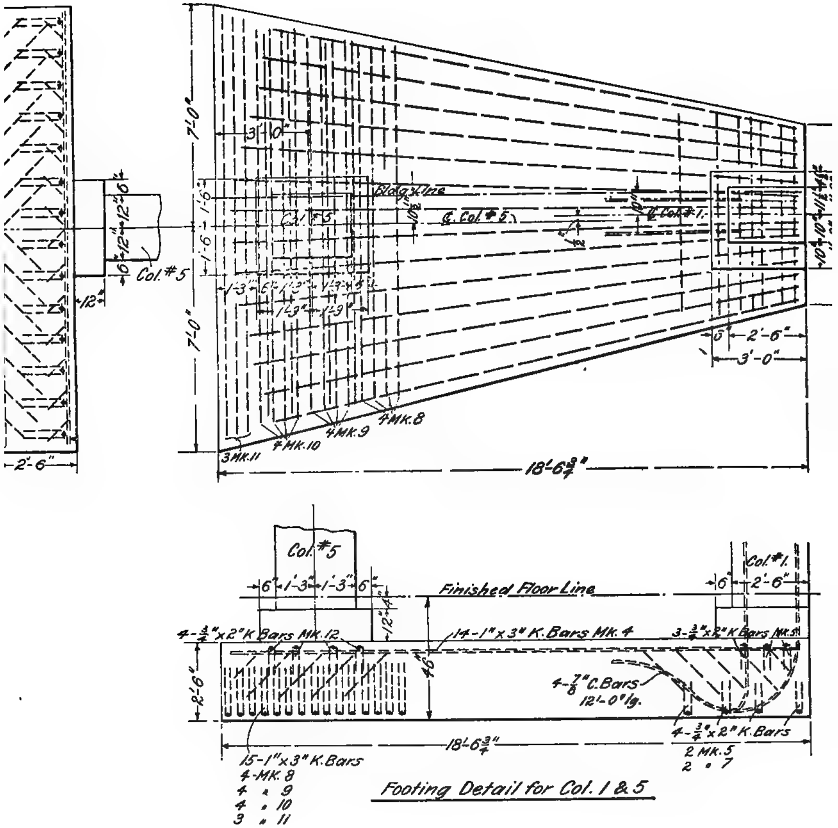
Section on d-d & e-e, 2d Floor



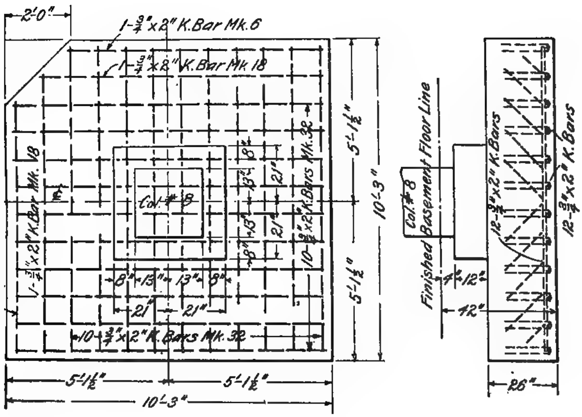




Section on d-d & e-e, 1st Floor

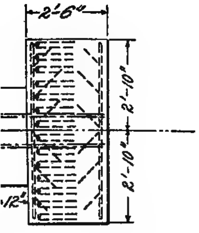


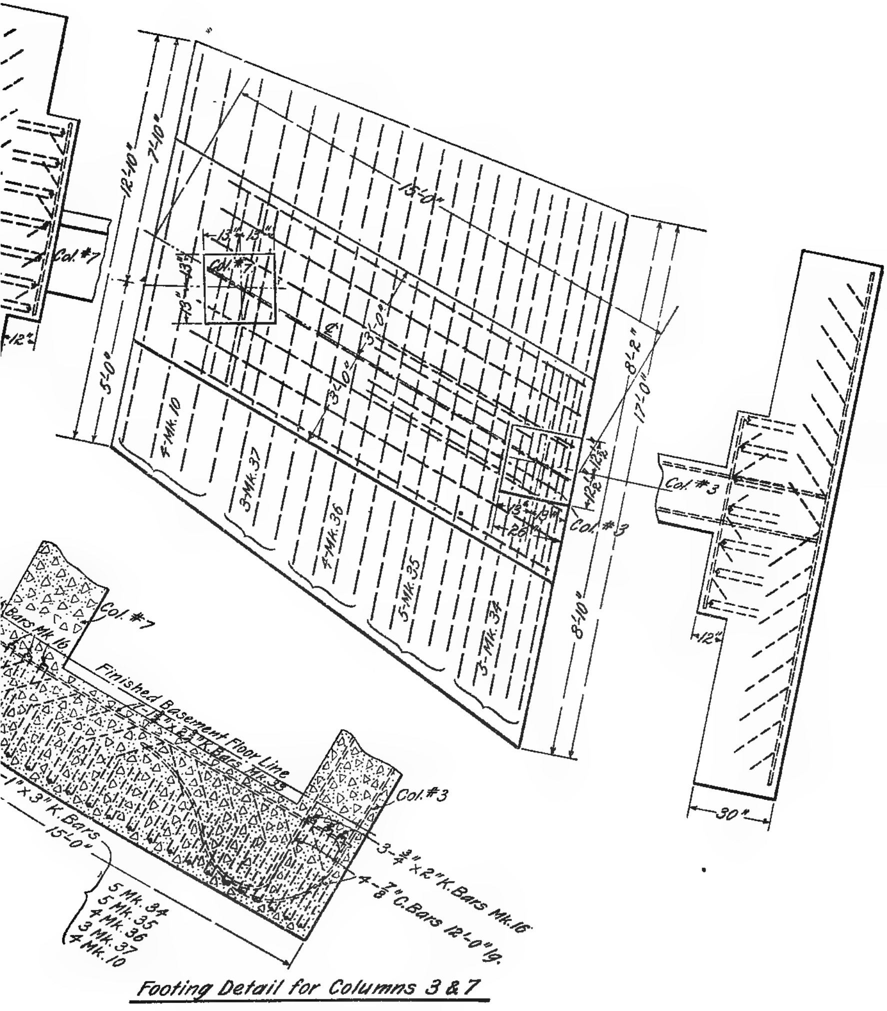
Footing Detail for Col. 1 & 5



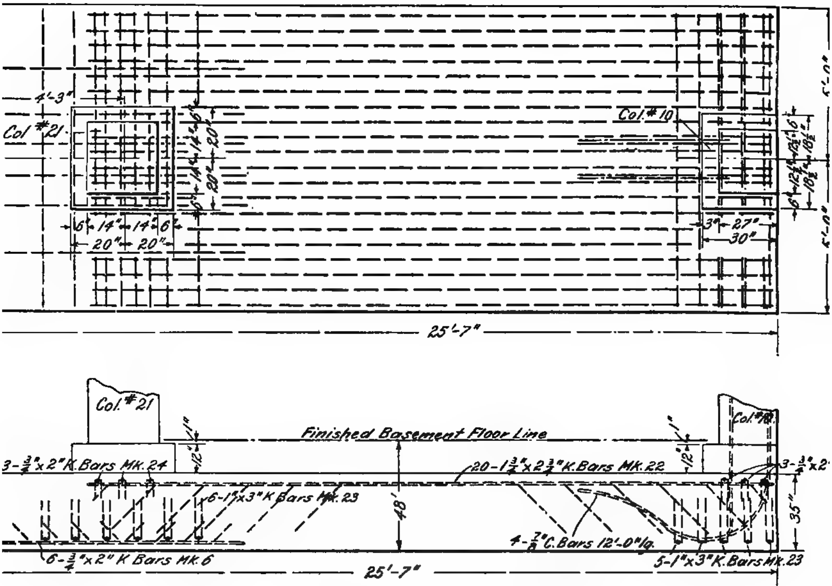
Secti

Footing Detail for Col. 8

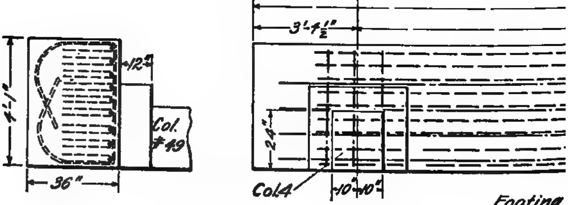




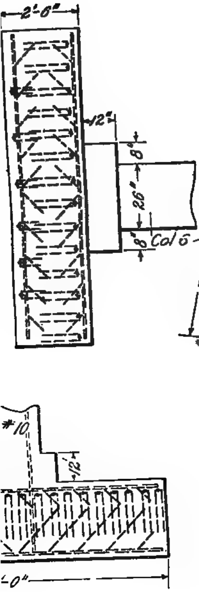
Footing Detail for Columns 3 & 7

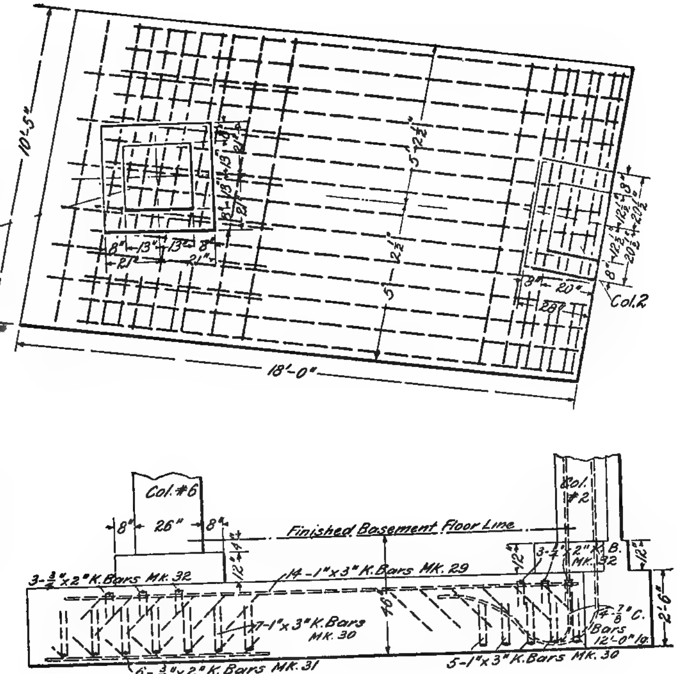


Footing Detail for Columns 10 & 21

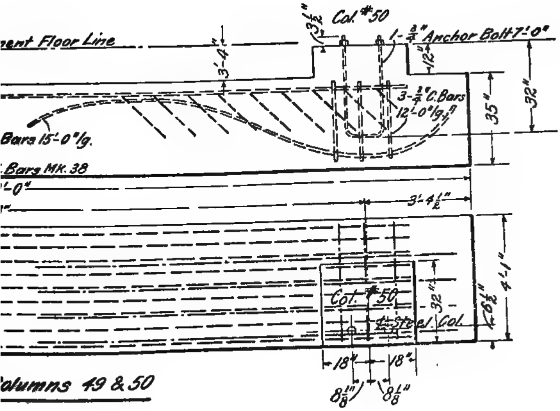


Footing

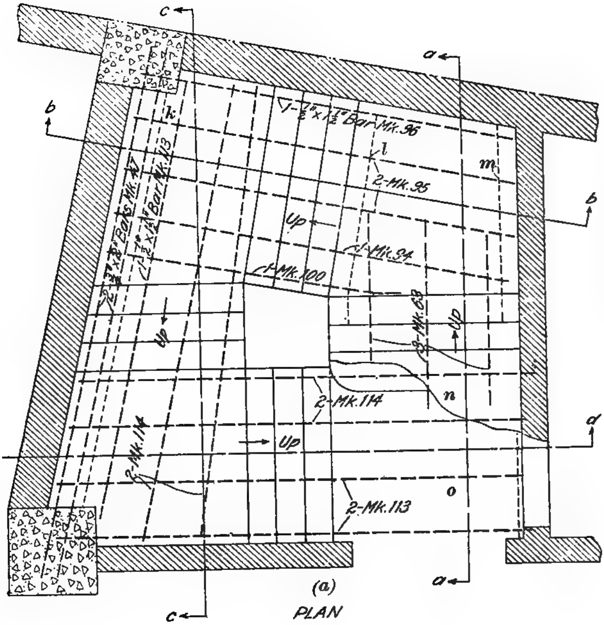




Footing Detail for Columns 2 & 6

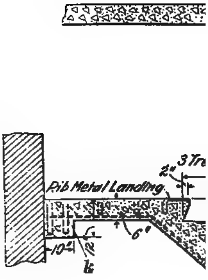


olumns 49 & 50

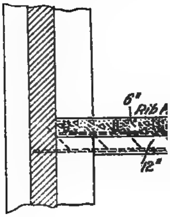


(a)

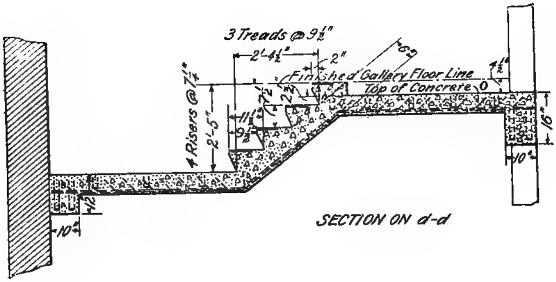
PLAN



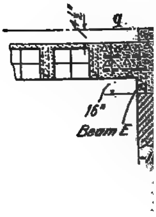
SECTION ON

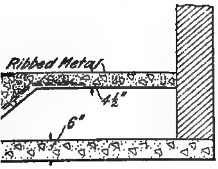


SEC

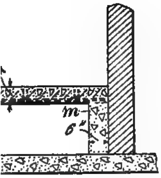


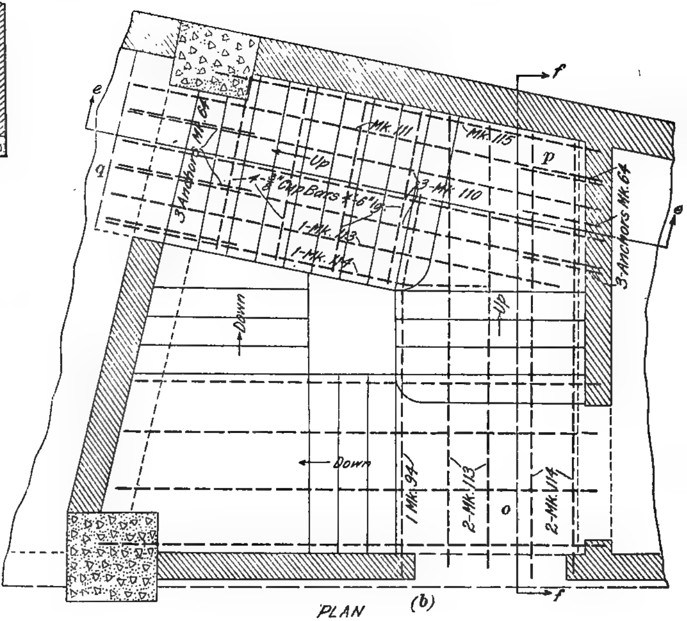
SECTION ON d-d



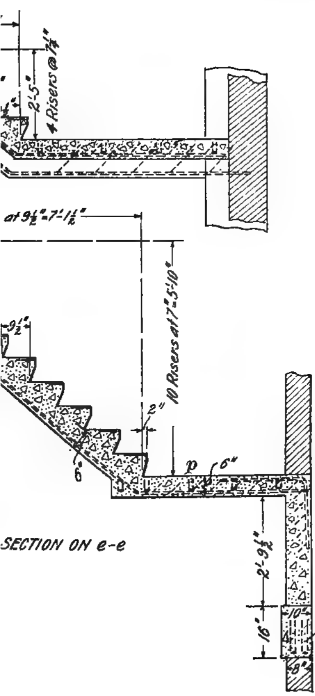


' ON a-a

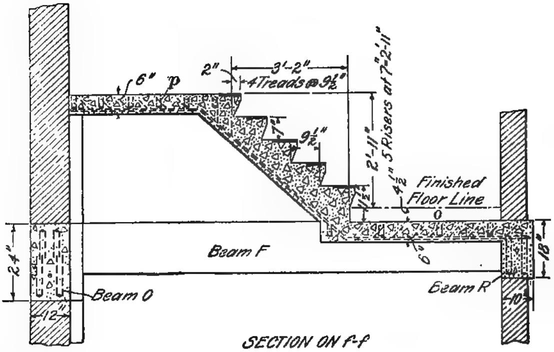




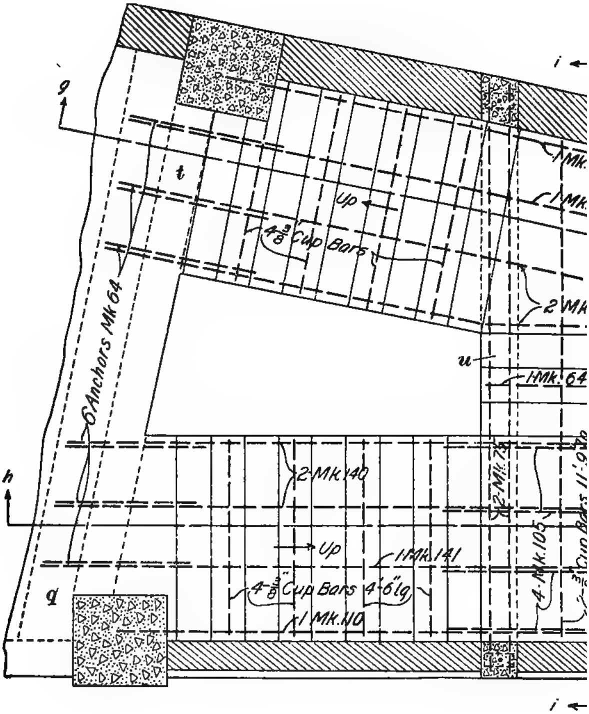
PLAN (b)



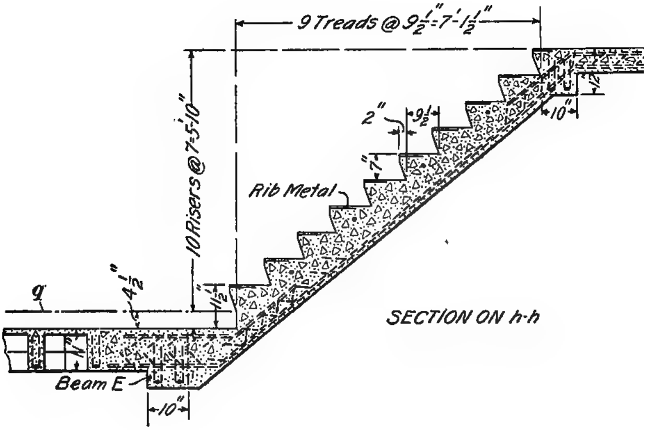
SECTION ON e-e



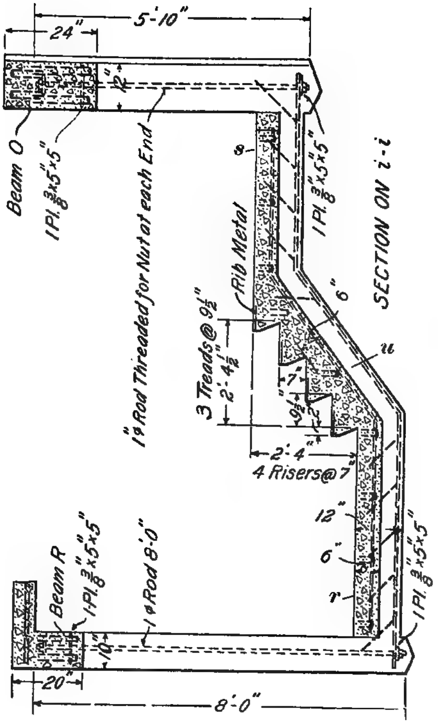
SECTION ON f-f

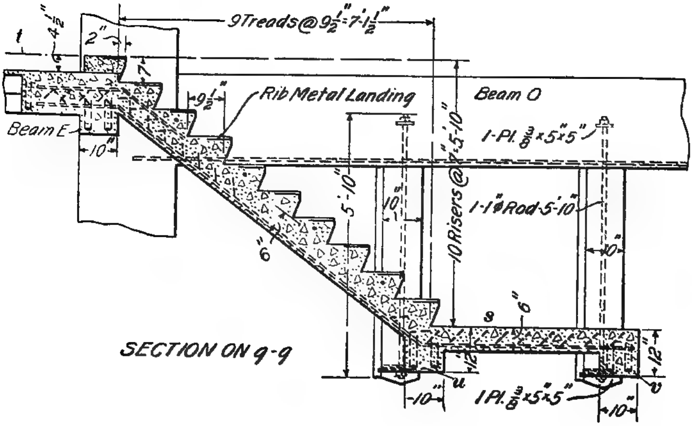


PLAN



SECTION ON h-h


SECTION ON i-i


SECTION ON g-g

The staircases, shown in Figs. 49 and 50, are those enclosed by beams *O*, *E*, *R*, and *F*, Fig. 45. In the plans of the staircases, the section lining on the walls between the concrete columns represents brick. The dotted lines *k*, *l*, and *m* in the plan, Fig. 49 (*a*), represent the sides *k*, *l*, and *m* of the beams shown in the section on *b–b*. In the plan (*a*), the lowest platform of the stairs is marked *n*, being on the same level as the basement floor. Proceeding along the stairs in the direction of the arrows, the platform *o*, which is on the level of the gallery floor, is finally reached. The various sections show the arrangement and dimensions of the stairs.

In the plan, Fig. 49 (*b*), the platform *o* is identical with that in plan (*a*). Proceeding from *o* to platform *p* and then to *q*, the floor level of the first floor is reached. In the plan, Fig. 50, *q* again represents the first-floor level, *r* and *s* intermediate platforms, and *t* the level of the second floor. Attention is called to the section on *g–g* and *i–i*, from which it is seen that the platforms *r* and *s* are suspended from the beams *R* and *O* by several 1-inch rods embedded in concrete suspension members, the latter being connected by means of two reinforced beams *u* and *v*. These members are built into the brick wall, as shown in the plan, Fig. 50.

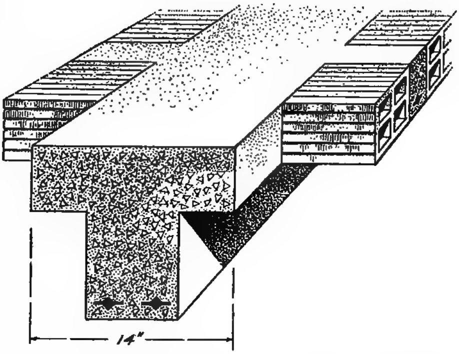


FIG. 53

BUILDING LAWS RELATING TO REINFORCED-CONCRETE CONSTRUCTION

50. As reinforced-concrete construction has become so extensively used, it has been necessary for cities, both in the United States and other countries, to adopt laws regulating the design and construction of reinforced-concrete buildings. Among the latest and most complete municipal regulations for reinforced-concrete construction are those recently adopted by the city of Philadelphia. These regulations have been prepared with considerable care, are wide in their scope, and at the same time are reasonable and practical. The regulations are given in the following articles:

51. The term *reinforced concrete* shall be understood to mean an approved concrete mixture, reinforced by steel or iron of any shape, so that the steel or iron will take up all the tensional stresses and assist in the resistance to compression and shear.

Before a permit to erect any reinforced-concrete structure is issued, complete specifications and drawings shall be filed with the Bureau of Building Inspection, showing all details of the construction, size and position of all reinforcing rods, stirrups, etc., and giving the composition and proportions of the concrete.

The execution of the work shall be performed by workmen under the direct supervision of a competent foreman or superintendent.

52. Reinforced-concrete construction will be accepted for fireproof buildings of the first class, if designed as hereinafter prescribed; provided, that the aggregate for such concrete shall be clean, broken, hard stone, or clean, graded gravel, together with clean, silicious sand or fine-grained gravel; should the concrete be used for flooring between rolled-steel beams, clean furnace clinkers entirely free of combustible matter, or suitable seasoned furnace slag may be used; when stone is used with sand or gravel it must be of a size to pass through a 1-inch ring, and 25 per cent. of the whole must not be more than one half the maximum size; and provided further, that the minimum thickness of concrete surrounding the reinforcing members of reinforced-concrete beams and girders shall be 2 inches on the bottom and 1½ inches on the sides of the said beams and girders. The minimum thickness of concrete under slab rods shall be 1 inch. All reinforcement in columns to have a minimum protection of 2 inches of concrete

53. All the requirements herein specified for the protection of steel and for fire-resisting purposes shall apply to reinforced-concrete flooring between rolled-steel beams, as well as to reinforced-concrete beams and to entire structures in reinforced concrete. Any concrete structure or the floor filling in same, reinforced or otherwise, which may be erected on a permanent centering of sheet metal, of metal lathing and curved bars or a metal centering of any other form, must be strong enough to carry its load without assistance from the centering, unless the concrete is so applied as to protect the centering as herein specified for metal reinforcement.

54. Exposed metal centering or exposed metal of any kind will not be considered a factor in the strength of any part of any concrete structure, and a plaster finish applied over the metal shall not be deemed sufficient protection unless applied of sufficient thickness and properly secured, as approved by the Chief of the Bureau of Building Inspection.

55. All concrete shall be mixed in a mechanical batch mixer to be approved by the Bureau of Building Inspection, except when limited quantities are required or when the condition of the work makes hand mixing preferable; hand mixing to be done only when approved by the Bureau of Building Inspection. In all mixing the material shall be measured for each batch.

When hand mixing is done under the aforesaid limitations, the cement and fine gravel or coarse sand shall be first thoroughly mixed dry and then made into a mortar by gradually adding the proper amount of water. The crushed stone or gravel shall be spread out to a depth not to exceed 6 inches, in a tight box or upon a proper floor, and be sprinkled with water as directed; the mortar is then to be evenly spread over the crushed stone, and the whole mass turned over a sufficient number of times to effect the thorough mixing of the ingredients.

56. All forms and centering for concrete shall be built plumb and in a substantial manner, made tight so that no part of the concrete mixture will leak out through cracks or holes, or joints, and after completion shall be thoroughly cleaned, removing shavings, chips, pieces of wood and other material, and no débris of any kind shall be permitted to remain in the forms. All forms to be properly supported and braced in a manner to safely sustain the dead load that may be imposed upon them during construction.

57. The reinforcing steel shall be accurately located in the forms and secured against displacement.

58. Concrete shall be placed immediately after mixing.

59. Whenever fresh concrete joins concrete that is set, the surface of the old concrete shall be roughened, cleaned, and spread with cement mortar, which mortar shall be mixed in proportions of 1 of cement to 2 of sand.

60. Concrete shall not be mixed or deposited in freezing weather, unless precautions are taken to avoid the use of materials covered with ice or that are in any other way unfit for use, and that further precautions are taken to prevent the concrete from freezing after being put in place. All forms under concrete so placed to remain until all evidences of frost are absent from the concrete and the natural hardening of the concrete has proceeded to the point of safety.

61. Concrete laid during hot weather shall be drenched with water twice daily, Sunday included, during the first week. The broken stone, if hot and dry, must be wet before going to the mixer.

62. The time at which props or shores may safely be removed from under floors and roofs will vary with the condition of the weather, but in no case should they be removed in less than 2 weeks; provided, that column forms shall not be removed in less than 4 days; provided further, that the centering from the bottom of slabs and sides of beams and girders may be removed after the concrete has set 1 week, provided, that the floor has obtained sufficient hardness to sustain the dead weight of the said floor and that no load or weight shall be placed on any portion of the construction where the said centers have been removed.

63. The concrete for all girders, beams, slabs, and columns, shall be mixed in the proportions of 1 of cement, 2 of sand or fine gravel, and 4 of other aggregates, as before provided. The concrete used in reinforced-concrete construction must be what is usually known as a *wet* mixture. When the concrete is placed under water it must be placed in a semidry state.

64. Only Portland cement shall be permitted in reinforced-concrete constructed buildings. All cement shall be tested, in carload lots when so delivered or in quantities equal to same, and report filed with the Bureau of Building Inspection before using it in the work. Cement failing to meet the requirements of the accelerated test will be rejected.

65. *Soundness, Accelerated Test.*—Pats of neat cement will be allowed to harden 24 hours in moist air, and then be submitted to the accelerated test as follows: A pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel, for 3 hours, after which, before the pat cools, it is placed in the boiling water for 5 additional hours.

To pass the accelerated test satisfactorily, the pats shall remain firm and hard, and show no signs of cracking, distortion, or disintegration.

66. Such cements, when tested, shall have a minimum tensile strength as follows: Neat cement shall, after 1 day in moist air, develop a tensile strength of at least 150 pounds per square inch; and after 1 day in air and 6 days in water shall develop a tensile strength of at least 500 pounds per square inch; and after 1 day in air and 27 days in water shall develop a tensile strength of at least 600 pounds per square inch. Cement and sand tests composed of 1 part of cement and 3 parts of crushed quartz shall, after 1 day in air and 6 days in water, develop a tensile strength of at least 175 pounds per square inch, and after 1 day in air and 27 days in water shall develop a tensile strength of at least 240 pounds per square inch. These and other tests as to fineness, set, etc., made in accordance with the standard method prescribed by the American Society of Civil Engineers, may, from time to time, be required by the Bureau of Building Inspection.

67. *Walls.*—Reinforced concrete may be used in place of brick and stone walls, in which cases the thickness may be two-thirds of that required for brick walls, as shown in the Schedule, Section 18 of the Act of Assembly, No. 123, of the Commonwealth of Pennsylvania, approved June 5, 1901, provided the unit stresses as set forth in these regulations are not exceeded.

Concrete walls in such cases must be reinforced in both directions in a manner to meet the approval of the Chief of the Bureau of Building Inspection.

68. *Steel.*—All reinforcements used in reinforced concrete shall be of standard grade of structural steel or iron of either grade to meet the Manufacturers' Standard Specifications, revised February 3, 1903.

69. Reinforced-concrete slabs, beams, and girders shall be designed in accordance with the following assumptions and requirements:

(*a*) The common theory of flexure to be applied to all beams and members resisting bending.

(*b*) The adhesion between the concrete and the steel is sufficient to make the two materials act together.

(*c*) The design shall be based on the assumption of a load four times as great as the total load (ordinary dead load plus ordinary live load).

(*d*) The steel to take all the tensile stresses.

(*e*) The stress-strain curve of concrete in compression is a straight line.

(*f*) The ratio of the moduli of elasticity of concrete to steel:

Stone or gravel concrete	1 to 12
Slag concrete	1 to 15
Cinder concrete	1 to 30

The allowable unit tranverse stress upon concrete in compression:

	POUNDS PER SQUARE INCH
Stone or gravel concrete	600
Slag concrete	400
Cinder concrete	250

The allowable unit stress in tension:

	POUNDS PER SQUARE INCH
Iron	12,000
Steel	16,000

The allowable unit shearing strength upon concrete:

	POUNDS PER SQUARE INCH
Stone or gravel concrete	75
Slag concrete	50
Cinder concrete	25

The allowable unit adhesive strength of concrete:

	POUNDS PER SQUARE INCH
Stone or gravel concrete	50
Slag concrete	40
Cinder concrete	15

The allowable unit stresses upon concrete in direct compression in columns:

	POUNDS PER SQUARE INCH
Stone or gravel concrete	500
Slag concrete	300
Cinder concrete	150

70. The allowable unit stress upon hoop columns composed of stone or gravel concrete shall not be over 1,000 pounds per square inch, figuring the net area of the circle within the hooping. The percentage of longitudinal rods and the spacing of the hoops to be such as to permit the concrete to safely develop the above unit stress with a factor of safety of 4.

71. When steel or iron is in the compression sides of beams, the proportion of unit stress taken by the steel or iron shall be in the ratio of the modulus of elasticity of the steel or iron to the modulus of elasticity of the concrete; provided that the rods are well tied with stirrups connecting with the lower rods of the beams; provided, further, that when rods are used in compression, the approval of the Chief of the Bureau of Building Inspection must be obtained.

72. In the design of structures involving reinforced-concrete beams and girders, as well as slabs, the beams and girders shall be treated as **T** beams, with a portion of the slab acting as a flange in each case. The portion of the slab that may be used to take compression shall be dependent upon the horizontal shearing stress that may exist in the beam, and in no case shall the slab portion exceed twenty times the thickness of the slab.

73. All reinforced-concrete **T** beams must be reinforced against the shearing stress along the plane of junction of the rib and the flange, using stirrups throughout the length of the beam. Where reinforced-concrete girders carry reinforced-concrete beams, the portion of the floor slab acting as flange to the girder must be reinforced with bars near the top, at right angles to the girder, to enable it to transmit local loads directly to the girder and not through the beams, thus avoiding an integration of compressive stresses due to simultaneous action as floor slab and girder flange.

74. In the execution of the work in the field, work must be so carried on that the ribs of all girders and beams shall be monolithic with the floor slabs.

75. In all reinforced-concrete structures special care must be taken with the design of joints to provide against local stresses and secondary stresses due to the continuity of the structures.

76. Shrinkage and thermal stresses shall be provided for by the introduction of steel.

77. In the determination of bending moments due to the external forces, beams and girders shall be considered as simply supported at the ends, no allowance being made for continuous construction over supports. Floor slabs, when constructed continuously, and when provided with reinforcement at top of slab over the supports, may be treated as continuous beams, the bending moment for uniformly distributed loads being taken at not less than $\frac{W L}{10}$. In the case of square floor slabs, that are reinforced in both directions and supported as continuous beams on all four sides, the bending moment may be taken as $\frac{W L}{20}$. In floor slabs in juxtaposition to the walls of the building, which are acting as simple beams the bending moment shall be considered as $\frac{W L}{8}$ when reinforced in one direction. If the floor slab is square and reinforced in both directions, but acting as a simple beam, the bending moment shall be taken as $\frac{W L}{16}$.

78. When the shearing stresses developed in any part of a reinforced-concrete building exceed the shearing strength as fixed in this section, a sufficient amount of steel shall be introduced in such a position that the deficiency in the resistance to shear is overcome.

79. When the safe limit of adhesion between the concrete and steel is exceeded, provision must be made for transmitting the strength of the steel to the concrete.

80. Reinforced concrete may be used, with the unit stresses previously given, for columns in which the ratio of the length to least side or diameter does not exceed fifteen. If more than fifteen diameters, the allowable stress shall be decreased proportionally. Reinforcing rods that are introduced for lateral stresses must be tied together at intervals of not more than the least side or diameter of the columns.

Longitudinal reinforcing rods will not be considered as taking any direct compression.

81. The contractor must be prepared to make load tests in any portion of a reinforced-concrete building within a reasonable time after erection, and as often as may be required by the Chief of the Bureau of Building Inspection. The tests must show that the construction will sustain a load equal to twice the calculated live load without signs of cracks.

ENGINEERING STRUCTURES IN REINFORCED CONCRETE

EXAMPLES OF ENGINEERING STRUCTURES

RESERVOIRS AND SIMILAR STRUCTURES

1. In the construction of reservoirs, especially those of limited capacity, such as are used to store water for fire-protection systems, reinforced concrete is extensively employed. Considerable use is also made of this material in the building of swimming pools.

In designing reinforced-concrete reservoirs, three important results must be accomplished: (1) the reservoir must be entirely water-tight; (2) it must be sufficiently strong to resist any failure that may be caused by the hydrostatic pressure of water contained in it; and (3) its walls must possess sufficient resistance to prevent it from collapse due to the pressure of the earth filling around the sides when the reservoir is empty.

Usually, when a reservoir is strong enough to resist the hydrostatic pressure of the water, it will offer sufficient resistance to the pressure of the earth acting against the outside of the walls. Some designers of concrete reservoirs consider the concrete as merely a lining, and do not design it to resist the pressure of the water, but depend on the earth filling, or embankments surrounding it, to provide the necessary stability. If a reservoir or tank structure is constructed

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in an excavation, and earth is afterwards filled in around it, the earth cannot be considered as offering much resistance in backing up the reinforced-concrete construction. In such cases, on account of the earth being merely fill, it will not pack solidly, and will shrink away from the concrete so that sufficient movement may occur to crack the side walls of the reservoir. In all instances, therefore, except where strictest economy is to be employed, the sides of a reservoir, a tank, or a swimming pool that is sunk in the ground should be carefully designed to resist the entire pressure of the water.

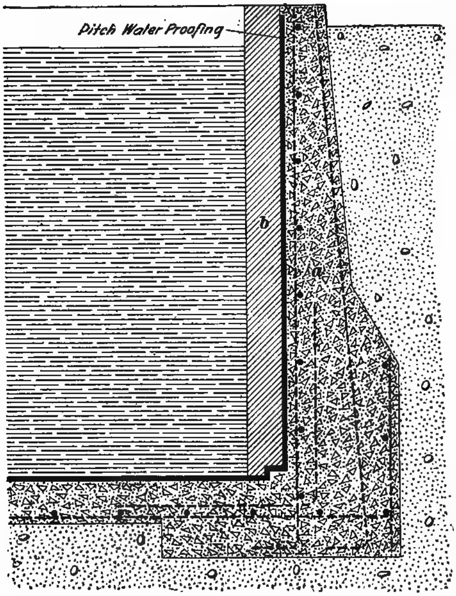


Fig. 1

2. Waterproofing of Concrete Reservoirs and Swimming Pools. The precautions to be taken to make reservoirs or swimming pools water-tight depend entirely on the special requirements of the case in hand. If small leakage or seepage through the walls of a reservoir is not important, it may be built of a good, rich, wet mixture of concrete with fine aggregates. Ordinarily, such a reservoir, if well backed up with clay puddle, will prevent any serious leakage or loss. However, if a reservoir is to be placed within a building and close to foundations, every precaution should be taken to make it absolutely water-tight.

The method of constructing an absolutely water-tight reservoir is illustrated in detail in Fig. 1. As shown at *a*, the sides and bottom are made of reinforced concrete, and next to this material, at both the bottom and the sides, is a coating of bituminous waterproofing laid as explained in a subsequent Section. The sides are protected by a 9-inch brick wall, as shown at *b*, laid up in cement mortar. This brick wall also holds the waterproofing in place.

It is usually necessary in such constructions to have a wall at least 9 inches in thickness, but where it is desired to decrease the cost, a 4-inch lining may be used. In such cases, wire wall ties must be built in the reinforced-concrete work so as to bond with the brickwork. Such ties must be provided for every 3 square feet of wall. The ties do not interfere with the waterproofing, because this material is put in hot and flows around them, making a tight junction.

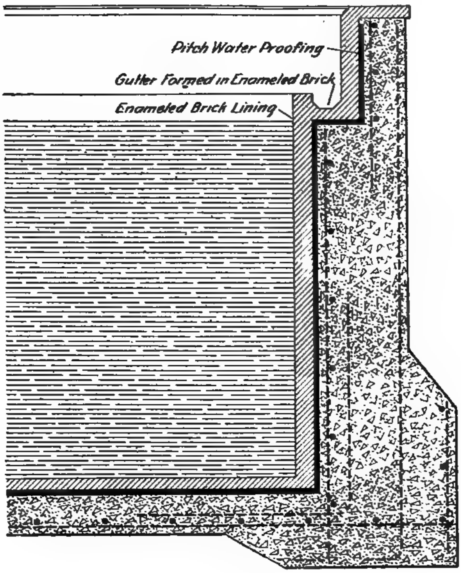
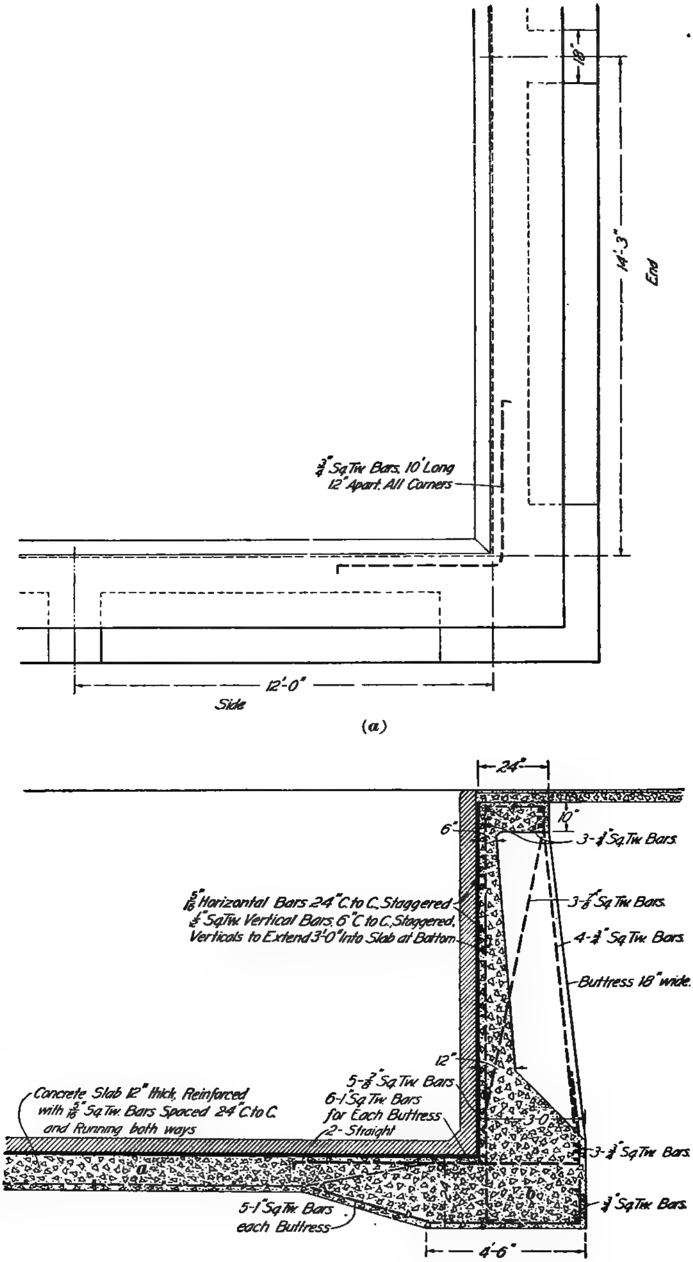


FIG. 2

3. It is always desirable to line swimming pools with enameled brick, tile, or marble, and no better construction can be suggested than the one illustrated in Fig. 2, in which white enameled brick is used for the inside finish of the pool. However, the cost of brick lining, together with the cost of the waterproofing, makes such structures expensive.

Reinforced-concrete reservoirs may be made practically water-tight by mixing about 5 per cent. of hydrated lime with the cement. The action of the lime is purely mechanical; it fills all the pores in the concrete and prevents leakage.


18"
14'-3"
End
3/4" Sq.Tw. Bars. 10' Long
12" Apart. All Corners
12'-0"
Side
24"
10"
6"
3-3/4" Sq.Tw. Bars.
5/16" Horizontal Bars 24" C.to C., Staggered
1/2" Sq.Tw. Vertical Bars, 6" C to C., Staggered,
Verticals to Extend 3'-0" Into Slab at Bottom
3-7/8" Sq.Tw. Bars.
4-3/4" Sq.Tw. Bars.
Buttress 18" wide.
12"
5-7/8" Sq.Tw. Bars
6-1" Sq.Tw. Bars
for Each Buttress
2-Straight
Concrete Slab 12" thick, Reinforced
with 5/16" Sq.Tw. Bars Spaced 24" C.to C.
and Running both ways
3-0
3-3/4" Sq.Tw. Bars.
a
b
3/4" Sq.Tw. Bars.
5-1" Sq.Tw. Bars
each Buttress
4'-6"

(a)

(b)

FIG. 3

4. Structural Design of Concrete Reservoirs and Swimming Pools.—In Fig. 3 is shown a plan and a section of a reinforced-concrete swimming-pool wall. This wall is designed to resist a 10-foot head of water and the pressure of the earth on the outside of the wall when the pool is empty. As shown at *a*, in section (*b*), the bottom consists of a 12-inch slab of concrete reinforced with $\frac{5}{16}$-inch square twisted bars. These bars are placed on 24-inch centers and extend in both directions. As the weakest part of a reservoir is in the corners where the vertical walls join the bottom, the concrete is thickened at this point, as shown at *b*.

Referring to the plan (*a*), it will be seen that the wall is arranged with buttresses, and that heavy buttresses are provided at the corners. The slab between these buttresses is comparatively thin, being 6 inches at the top and 12 inches at the bottom. The pool is reinforced around the top with a rectangular sill that connects the buttresses and thus adds to the strength of the wall. The buttresses are 18 inches in width and are strongly reinforced to resist both the pressure of the water from the inside and the pressure of the earth from the outside when the pool is empty. The slab forming the wall construction between the buttresses is reinforced with horizontal shrinkage bars and with vertical bars of $\frac{1}{2}$-inch square twisted steel placed 6 inches from center to center. As will be observed from the section, the reinforcing bars in the buttress interlace with heavy bars running into the bottom of the reservoir, the combination of bars forming a strong reinforcement about the bottom corner and tending to prevent the wall from turning about this point.

5. Tanks Supported at the Top.—If a reservoir with the walls tied in at the top and the bottom is to be constructed, a very economical design can be adopted. In Fig. 4 is shown a reservoir of this type designed to hold the main water supply for a fire-protection system. As the reservoir is to be sunk in the ground beneath a building, a reinforced-concrete floor construction is arranged at the top, being level with the floor of the building. In this way, the reinforced-concrete girders and beams forming the floor construction securely tie in and brace the top of the reservoir wall against failure from internal or external pressure, and the heavy reinforced-concrete bottom prevents the wall from moving at this point. As will be observed, the reservoir is waterproofed and the vertical walls are protected with a 9-inch brick lining. The rods in the bottom of the reservoir are extended in both directions to provide against shrinkage, and are turned up at the ends to bond with the reinforcement

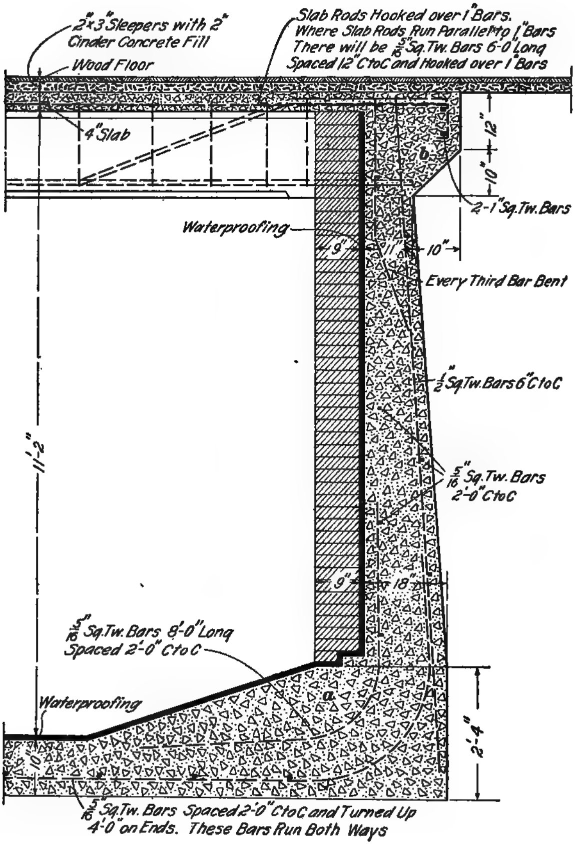

2"×3" Sleepers with 2" Cinder Concrete Fill
Wood Floor
Slab Rods Hooked over 1" Bars. Where Slab Rods Run Parallel to 1" Bars There will be 5/16" Sq. Tw. Bars 6-0" Long Spaced 12" C to C and Hooked over 1" Bars
4" Slab
12"
10"
b
2-1" Sq. Tw. Bars
Waterproofing
9"
11"
10"
Every Third Bar Bent
1/2" Sq. Tw. Bars 6" C to C
5/16" Sq. Tw. Bars 2'-0" C to C
11'-2"
9"
18"
5/16" Sq. Tw. Bars 8'-0" Long Spaced 2'-0" C to C
Waterproofing
a
10"
2'-4"
5/16" Sq. Tw. Bars Spaced 2'-0" C to C and Turned Up 4'-0" on Ends. These Bars Run Both Ways

Fig. 4

of the wall. The bottom corner of the reservoir wall, as at *a*, is made very secure by sloping up the bottom and thickening the concrete at this point; besides, additional reinforcement consisting of $\frac{5}{16}$-inch square twisted bars placed 2 feet from center to center is provided at this point.

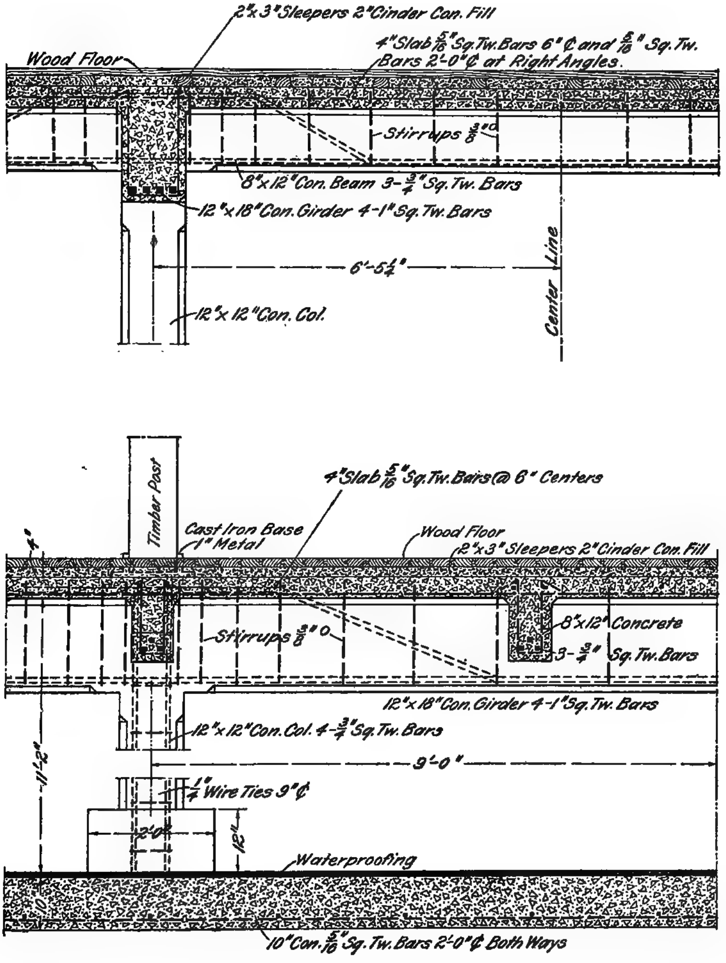

2"x 3" Sleepers 2" Cinder Con. Fill
4" Slab 5/16" Sq. Tw. Bars 6" ₵ and 5/16" Sq. Tw. Bars 2'-0" ₵ at Right Angles.
Wood Floor
Stirrups 3/8" ○
8"x12" Con. Beam 3-3/4" Sq. Tw. Bars
12"x18" Con. Girder 4-1" Sq. Tw. Bars
6'-5 1/4"
Center Line
12"x12" Con. Col.
Timber Post
4" Slab 5/16" Sq. Tw. Bars @ 6" Centers
Cast Iron Base 1" Metal
Wood Floor
2"x3" Sleepers 2" Cinder Con. Fill
4"
Stirrups 3/8" ○
8"x12" Concrete 3-3/4" Sq. Tw. Bars
12"x18" Con. Girder 4-1" Sq. Tw. Bars
12"x12" Con. Col. 4-3/4" Sq. Tw. Bars
9'-0"
1/4" Wire Ties 9" ₵
2'0"
12"
11'-2"
10"
Waterproofing
10" Con. 5/16" Sq. Tw. Bars 2'-0" ₵ Both Ways

FIG. 5

In calculating the thickness of the wall, it is considered as a slab supported at the top and the bottom. As the pressure of the water increases toward the bottom, so that, diagrammatically, the variation in load would be represented by a triangle, the wall must be thicker at the bottom than at the top. This is accomplished by keeping the wall straight on the inside and battering it on the outside. The reservoir is somewhat strengthened by extending a rim of concrete around the top and reinforcing it in the manner shown at *b*, also, the reinforcement of the floor system securely bonds and ties in with the concrete of the wall.

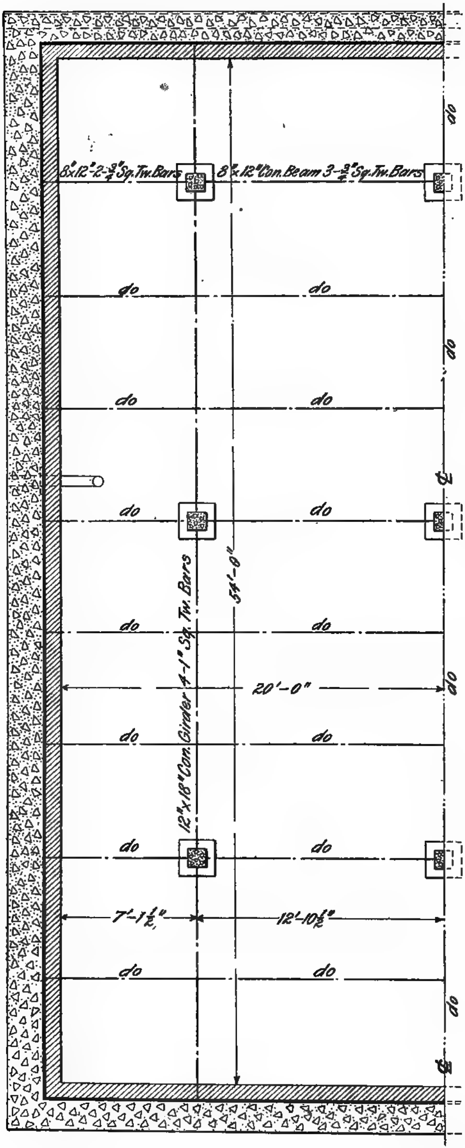


FIG. 6

Details of the floor construction and the method of arranging the column supports of the building over the reservoir just described are shown in Fig. 5. The size of the reservoir, as well as the framing plan of the floor construction, is shown in the half plan, Fig. 6.

6. Roofs Over Reservoirs.—Large reservoirs for town water supplies are sometimes roofed over with reinforced concrete. Such constructions usually consist of heavy columns supporting a reinforced-concrete slab without beams or girders. This type of construction is shown in Fig. 7, which illustrates the roof of the reservoir of the Massachusetts State Farm, at Bridgewater, Massachusetts. As will be observed, the concrete slab is reinforced with strips of expanded metal that run in both directions across the column; besides this, other reinforcement is provided throughout the balance of the slab. The several sections show clearly the strips used over the top of the columns to obtain the advantage of continuous reinforcement. The reinforcement used throughout this construction consists of 3-inch expanded metal.

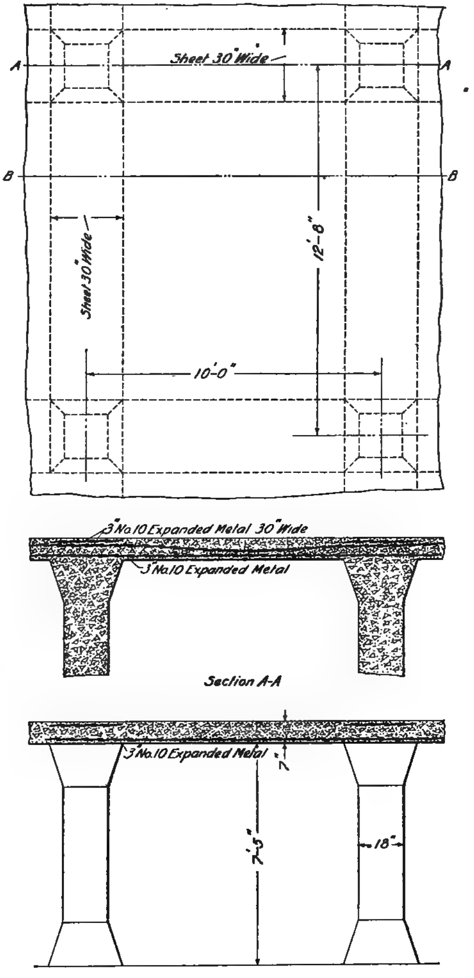


FIG. 7

REINFORCED-CONCRETE TANK-TOWER CONSTRUCTION

7. In many towns and small cities, the water-supply tanks are supported by reinforced-concrete towers of the type shown in Fig. 8. It has frequently been found that such towers can be built as cheaply as can steel structures; besides, they have the advantage of requiring no painting from time to time and are exempt from rust, corrosion, or deterioration. When designed with the intention of improving the appearance of the structure, a reinforced-concrete tower can be more easily decorated than can a tank support made of steel.

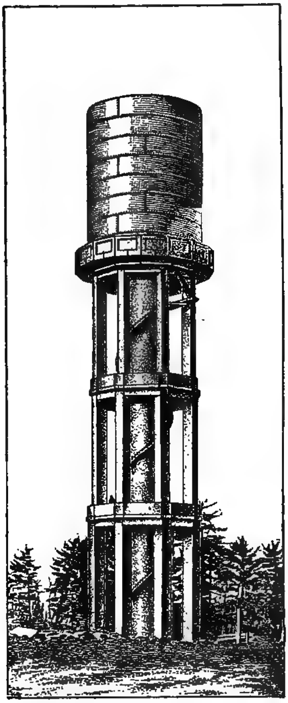


FIG. 8

The tower shown in Fig. 8 was designed to carry a 200,000-gallon steel tank. The tower consists of eight concrete columns spaced on centers around the circumference of a 26½-foot circle and surrounding a hollow concrete cylinder with an inside diameter of 8 feet. The columns and cylinder are held together by two intermediate platforms and a heavy platform, or slab, at the top. The footing is spread over the entire base of the tower, and is in the form of a sixteen-sided polygon, being 6 feet in thickness and 38 feet in diameter at the bottom. The central shaft, in addition to carrying some of the load of the tank, acts as a cylinder around which the reinforced-concrete stairs are run and likewise forms a shaft for the pipes leading to and from the tank. The columns of the lower tier are 3 feet 6 inches square; those of the second tier, 3 feet square; and those of the top tier, 2 feet 6 inches square. The offset is made at the outside of the column, as shown in Fig. 9.

8. The details of construction and the method of reinforcing the columns, cylinder, and balcony floor construction are illustrated in Fig. 10, a half plan of the structure being shown at (*a*), and a section through the column construction, balcony, and cylinder, at (*b*). As will be observed, the thickness of the cylinder is dropped off 3 inches at each tier; thus, the walls of the cylinder at the bottom are 18 inches in thickness, and at the second and the third tier they are 15 inches and 12 inches, respectively. The reinforcing rods are likewise reduced in size in both the columns and the cylinder from the first tier upwards. In the lower tiers, the columns have four 1⅞-inch round rods; in the second tier, four 1⅝-inch rods; and in the upper tier, four 1⅜-inch rods. The vertical reinforcing rods are placed near the corners of the columns and are tied in with ¼-inch wire ties placed 12 inches from center to center. These rods were figured in estimating the compressive strength of the columns. Their ends are neatly fitted and are surrounded with a pipe sleeve. The bottom rods project into the reinforced-concrete footing and are slipped into pipe sleeves provided with a 12-inch circular cast-iron base. In this way, the bearing strength of the bars was well realized. The cylinder is reinforced with vertical rods placed about 12 inches from center to center, and horizontal reinforcement, consisting of 1-inch hoop rods, is also provided in it. The balcony floors are reinforced with rods that radiate from the cylinder and bear upon the concrete

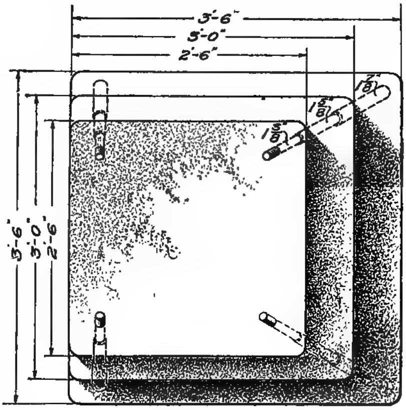


FIG. 9

lintels spanning the space between the columns, each of these lintels being reinforced with four corrugated bars. The floor

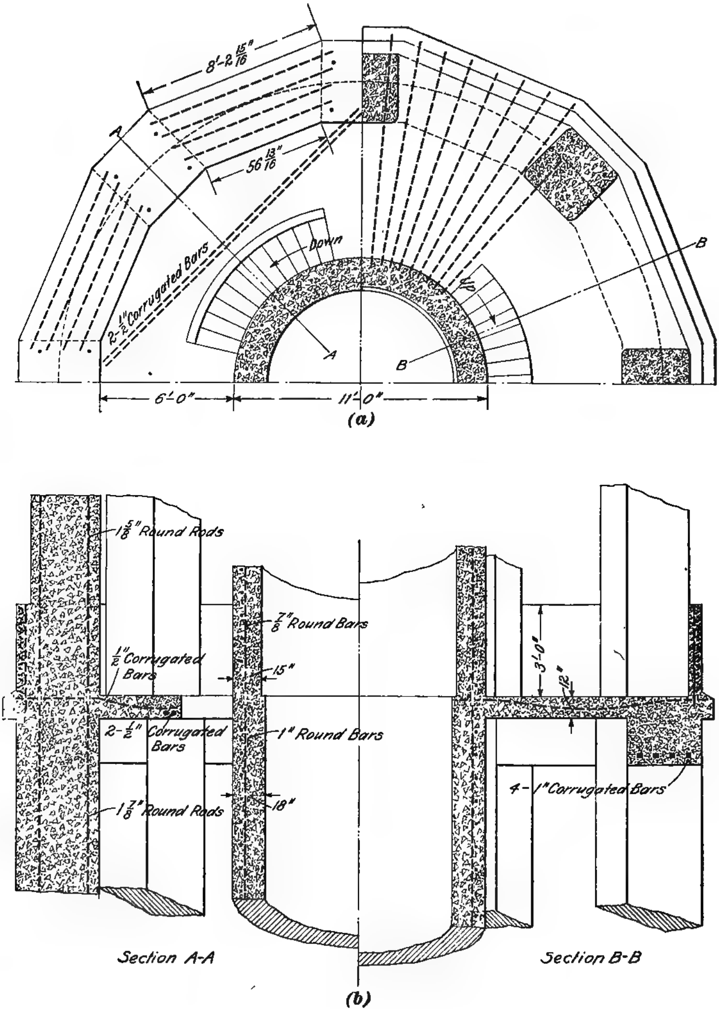


FIG. 10

of the third balcony, which carries the tank, is about 36½ feet in diameter and 16 inches in thickness. It is heavily reinforced with radiating 1-inch rods, and is cantilevered about 4 feet beyond the column. At each balcony is provided a monolithic railing, or balustrade. This railing is about 3 feet high and 6 inches in thickness and is reinforced.

9. In obtaining proposals for the construction of this tank tower, which is located at Bordentown, New Jersey, bids on a steel structure were also secured. In the eleven bids submitted, one contractor quoted the same price for both types of construction, four quoted a lower price for the concrete tower, and the other six were lowest for the steel tower. The contract price for the reinforced-concrete construction was $10,500. In the construction of this work, a 1–3–5 mixture was used for the foundation and a 1–2–4 mixture, placed very wet, was employed for the rest of the construction. The aggregates used throughout the construction were sand and good, clean gravel, which, being very coarse, took the place of broken stone. No effort was made to finish the structure after erection, but in placing the concrete a very wet mixture was used. It was thoroughly spaded along the sides of the forms, so as to force the gravel away from the surface and allow the neat cement to mold smooth against the form boards. The work that resulted was smooth and presentable.

REINFORCED-CONCRETE RETAINING WALLS

10. Walls of Moderate Height.—Both plain concrete and concrete reinforced with rods or bars may be used for the construction of retaining walls. In many instances, the proportions used for stone or masonry walls are adopted for concrete walls. If such walls are proportioned to resist the thrust of the earth filling, or backing, by their weight, their section will approach the proportion of a masonry wall.

A retaining wall of this type is shown in Fig. 11. Except where stone for masonry construction is not readily obtainable, this type of wall is generally more costly than a stone wall. The wall shown is not reinforced, but is molded in one mass; it will offer greater resistance than a masonry wall of the same section. The figure also shows the junction of a concrete wall with a masonry wall, which is arranged to be bonded to the concrete.

11. If a concrete wall is to be heavily reinforced with steel bars or rods, a very economical section may be employed. A typical reinforced-concrete wall is shown in Fig. 12. This wall is about 12 feet high, 14 inches thick at its junction with the base, and 8 inches thick at the top. The wall is so reinforced that the base and the wall will act as a monolith. The reinforcement for this purpose consists of the vertical rods indicated at *a*. Besides this reinforcement, the wall is provided with shrinkage rods or bars, as shown at *b*.

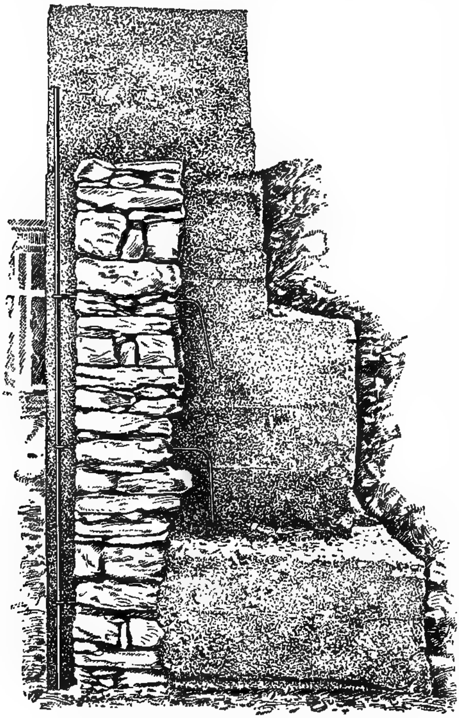


Fig. 11

12. High Retaining Walls. If retaining walls are to be over 12 feet in height, they are often designed with a buttress every 10 or 12 feet. In addition, the base or footing of the wall is so spread that the weight of the fill will partly rest on it. In this way, the resistance of the wall to overturning is increased materially by the weight of the earth filling back of it.

13. A design suitable for a high retaining wall is shown in Fig. 13. This wall is practically 12 inches in thickness throughout, as this is the minimum thickness that can well be molded when the forms are of a height as great as 12 feet. From the section it will be observed that the footing, which is of reinforced concrete and monolithic with the wall, is extended on both sides. In order to stiffen the wall and to cause it to act merely as a slab subjected to transverse stress between supports, the buttresses shown at *a* are provided. As the resultant pressure of the earth is in the direction shown by the arrow *b*, the buttresses are subjected to transverse stress as a cantilever. The compression area of the section is taken up by the wall, which acts in conjunction with the buttress proper to form a **T** section, and the tension at the inside edge of the buttress is provided for by Kahn bars, as shown at *c*.

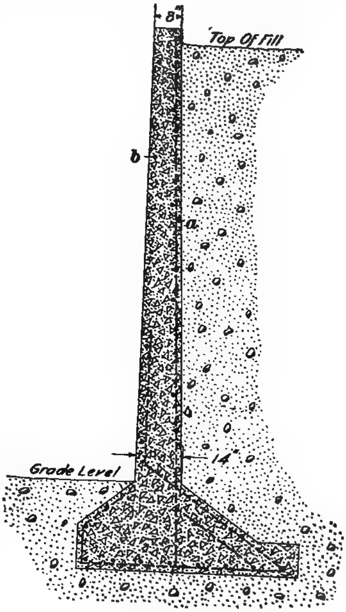
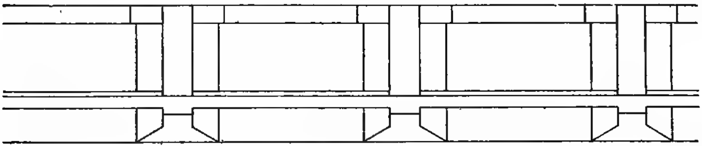


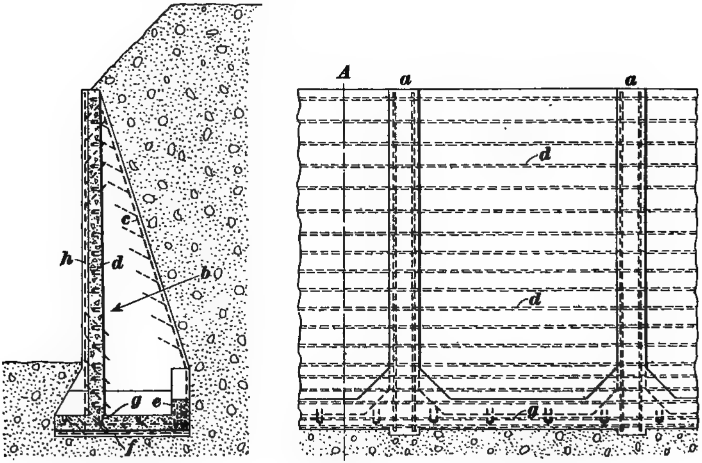
Fig. 12

The wall spans from buttress to buttress in the same manner as a reinforced floor slab spans between beams or girders. The wall is consequently reinforced with horizontal Kahn bars, as at *d*, and as the thrust of the earth per square foot on the back of the wall increases with the depth, these bars are placed closer together at the bottom, the spacing being increased toward the top.

To prevent the failure of the footing, which is made only about 16 inches thick, it is reinforced along the inside edge with the beam *e*. This beam is monolithic with the buttress, so that the footing is logically reinforced with transverse bars, as shown at *f*. In order that the entire design may be stiffened and strengthened, gussets or fillets are inserted in the junctions between the buttresses and the footing. To reinforce the footing further, longitudinal bars which act in the nature of double reinforcement are provided, as shown at *g*. Additional reinforcement *h* running in a vertical direction is provided in the wall. This reinforcement assists in taking up some of the compression on the compressive side of the buttress and tends further to prevent failure of the wall by shearing.



PLAN



SECTION AT A-B

REAR ELEVATION

Fig. 13

14. Retaining Walls of Great Height.—If retaining walls are to be of great height, say from 40 to 60 feet, a section designed as illustrated in Fig. 14 is frequently used. The action of this wall is similar to that of the wall just described. For this reason, the footing is extended in order to get a wide base to guard against overturning, and also in order that the weight of the earth filling will act in conjunction with the weight of the wall in resisting the horizontal pressure. By providing the upper tier, or bench, of concrete *a*, the wall is greatly stiffened throughout its length and the weight on the back of the wall is increased, because, in a high wall, when it tends to overturn, the footing at *b* would not get the benefit of the weight of the entire prism of earth extending from the footing to the top of the fill, for, in lifting, the earth would tend to form a triangular

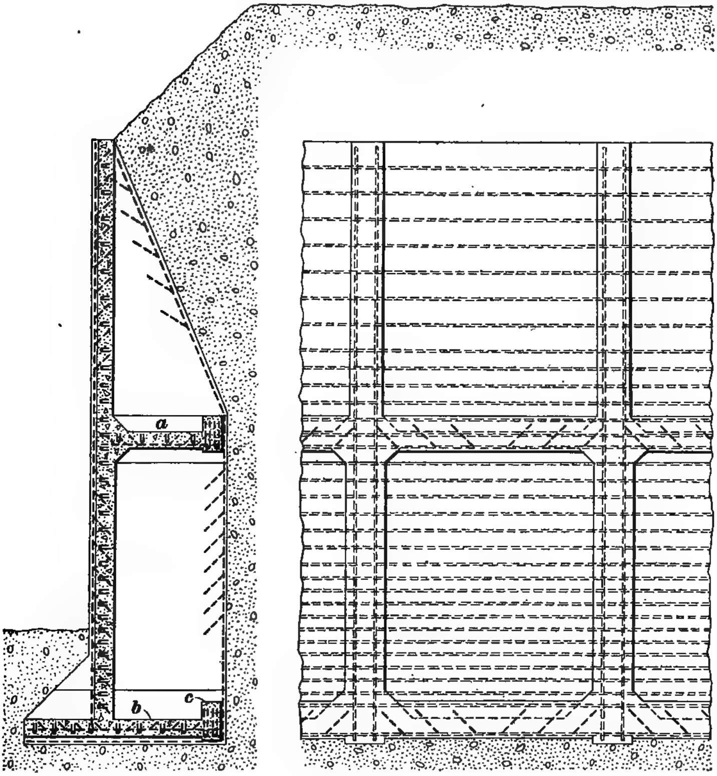


FIG. 14

wedge extending from the inside of the top of the wall to the inside edge of the footing. By providing the second tier, or bench, as at *a*, the stability of the wall is maintained by the weight of a prism of earth contained between the footing and the tier, or bench, and also by the earth above the tier.

This wall is reinforced with horizontal Kahn bars that strengthen the slab construction between the buttresses, and with buttress reinforcement that provides the necessary tensile resistance on the tension side of this section. The footing is reinforced in both directions, and is strengthened by the beam *c* extending between the buttresses.

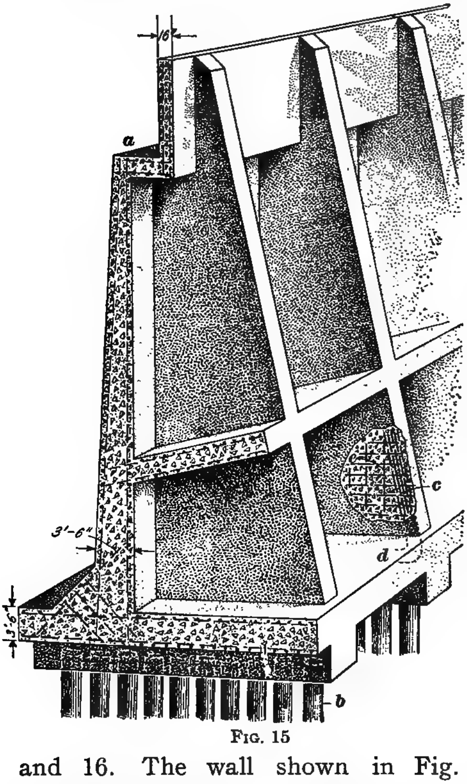


FIG. 15

15. Details of Construction and Reinforcement. Two types of reinforced-concrete retaining walls or abutments are shown in Figs. 15 and 16. The wall shown in Fig. 15 is about 50 feet in height, and is constructed so that a seat *a* is provided for a plate girder. As this retaining wall was built upon soft ground, the foundation was piled, as shown at *b*. As indicated in the figure, the wall is 3 feet 6 inches thick at the bottom and 16 inches thick at the top. It is reinforced with $\frac{3}{4}$-inch square twisted bars, placed 12 inches from center to center, extending horizontally, and with $\frac{1}{2}$-inch square twisted bars extending vertically. Shear bars, or lap bars, are provided in a horizontal direction at each abutment. These are 5 feet long and ¾-inch square twisted in section. The buttress is reinforced on the tension side with 1¼-inch square bars, as shown at *c*. In order to save metal, the bars are stopped off. Thus, in the lower third there are nineteen 1¼-inch bars; in the middle third there are eleven 1¼-inch bars; and extending into the upper third of the buttress there are only five 1¼-inch bars. The buttress is reinforced throughout horizontally by sets of ½-inch square twisted bars that interlace with the oblique rods at the inside edge of the buttress. As shown at *d*, the main reinforcing rods of the buttress are turned into the concrete of the footing. At about one-third of the height there is provided a transverse reinforced-concrete shelf. This shelf is strongly reinforced with transverse rods, those in the bottom extending the full distance between centers of buttresses. The rods in the upper part are 5 feet long, and reinforce the shelf at the bearings on the buttress in such a way as to produce the effect of a fixed beam. Vertical rods are also supplied in the buttresses. These consist of ¾-inch square bars running from the footings to the top of the buttresses, as shown at *e*. The footing is reinforced in both directions, at both top and bottom as will be noted by reference to the lower part of the illustration.

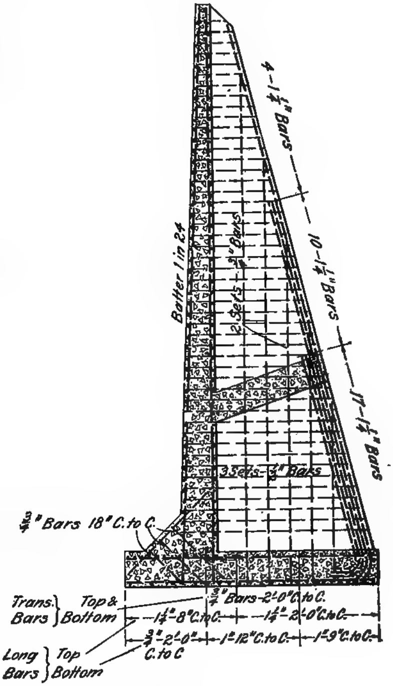


FIG. 16

16. The retaining wall shown in Fig. 16 is approximately the same height as the one just described. It has no shelf formed for the bearing of a girder, but is reinforced in much the same manner. It is an excellent design for retaining walls of great height and of economical construction. As such walls are built in sections, the maximum economy would be realized if the extent of the wall were considerable and the form work could be used several times.

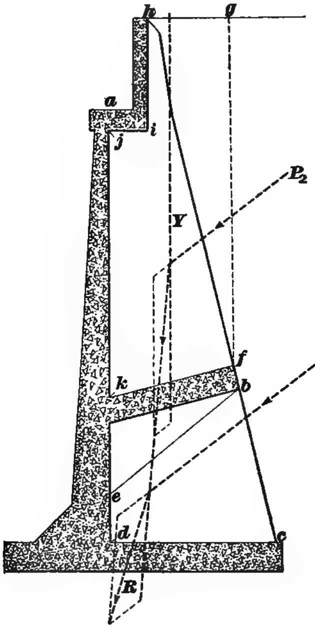
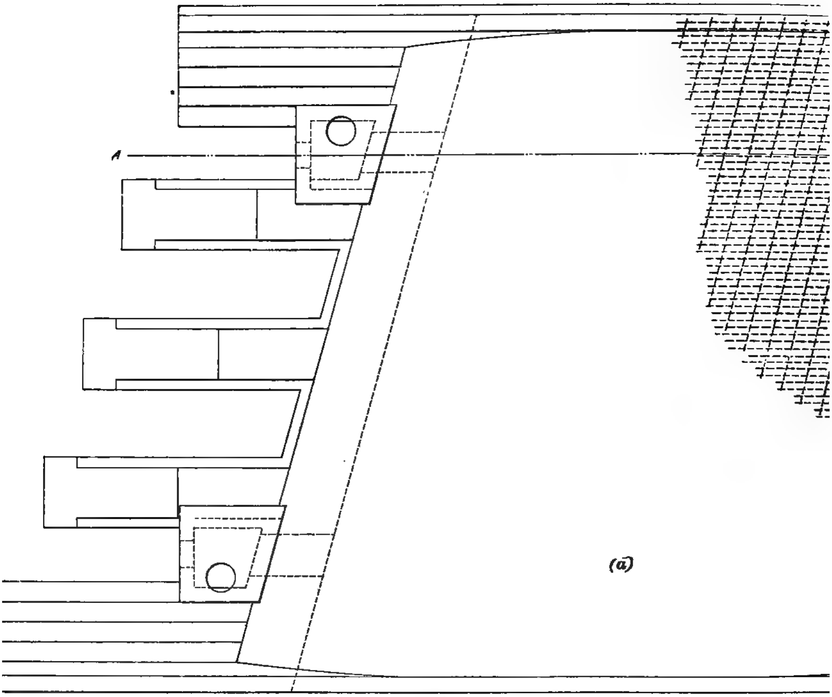
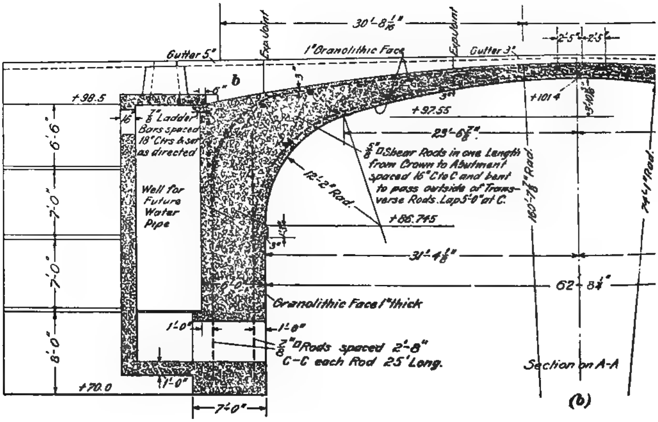


FIG. 17

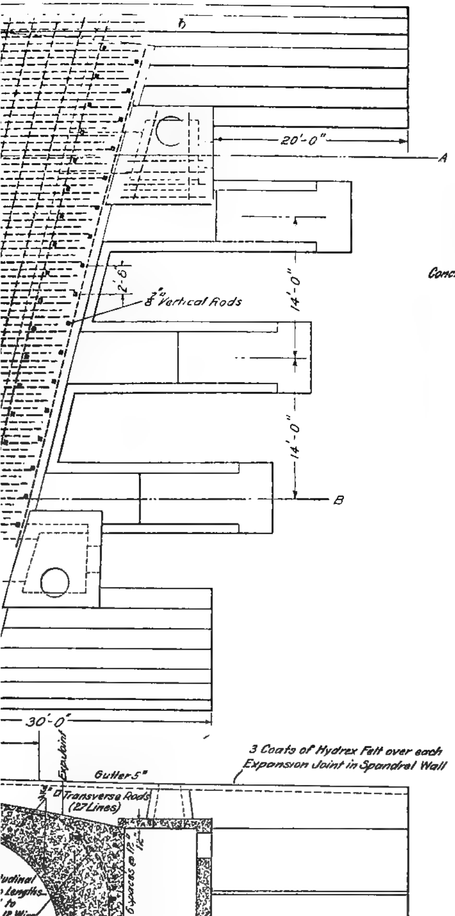
17. Operation of Forces. The analysis of the forces operating upon a retaining wall of the section shown in Fig. 15 is interesting, and is represented diagrammatically in Fig. 17. In the analysis of a wall of this kind, all the vertical forces, such as the reaction from the plate girder at *a*, and the weight of the several portions of the retaining wall and earth filling on the footing and projecting shelf, as *b c d e* and *f g h i j k*, respectively, must be found in their true position on the section and their resultant obtained. The position of the resultant is found by the theory of moments. After the resultant vertical force, as at *Y*, and its position have been determined, the introduction of the thrusts from the fill is necessary to produce the final resultant. It will be noticed that the pressure of the earth in the lower section of the wall, as at P_1, is separated from the pressure of the earth above the upper tier, which is shown at P_2. It is conservative practice to consider the pressure of the earth as applied in this manner. These several forces are combined in the final resultant, as shown at *R*, but if this resultant intersects the base of the wall within the middle third of its width, there is

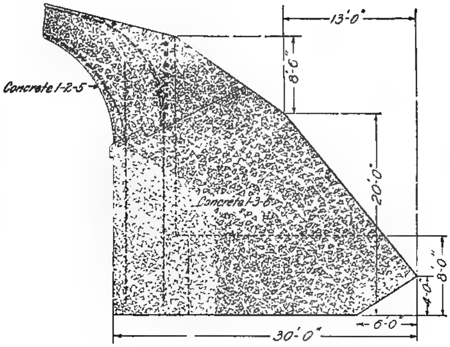

A

(a)


30'-8 1/16"
Exp. Joint
Exp. Joint
2'5"
2'5"
Gutter 5"
1" Granolithic Face
Gutter 3"
b
6"
3"
+98.5
16"
7/8" Ladder Bars spaced 18" Ctrs & set as directed
+101.4
+97.55
3"
23'-6 7/8"
5/8" □ Shear Rods in one Length from Crown to Abutment spaced 16" C to C and bent to pass outside of Transverse Rods. Lap 5'-0" at C.
12'-2" Rad.
160'-7 7/8" Rad.
74'-1" Rad.
6'-6"
7'-0"
7'-0"
8'-0"
Well for Future Water Pipe
+86.745
3"
31'-4 1/8"
62'-8 1/4"
6'-0"
Granolithic Face 1" thick
1'-0"
1'-6"
7/8" □ Rods spaced 2'-8" C-C each Rod 25' Long.
1'-0"
+70.0
7'-0"
Section on A-A

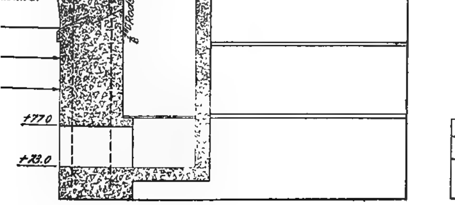
(b)

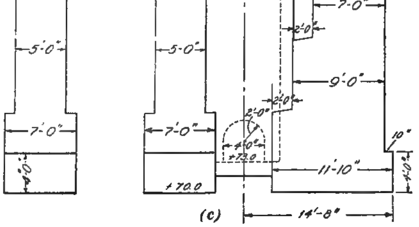




Section on B-B

(d)


3 Coats of Hydrex Felt over each Expansion Joint in Spandrel Wall
Expansion Joint
Gutter 5"
Transverse Rods (27 Lines)
...tudinal
...Lengths
... to
12 Wire.
3/4" Rods 6 spaces @ 1'2"
+77.0
+73.0



(c)

no tension on the inside edge of the footing, and the stresses vary from a minimum on the inside edge to a maximum at the outside edge. The principles for analyzing the forces tending to overturn a retaining wall and the resistance of the wall to them are described at length in *Retaining Walls*. This diagram is given merely to show the analysis necessary for the design of the constructions illustrated in Figs. 15 and 16.

REINFORCED-CONCRETE BRIDGES

18. Bridge of Moderate Span.—Many bridges over streams and roadways are now constructed of reinforced concrete, and if a durable structure is required at a moderate cost, and also if appearance is a factor to be considered, there is no better material. In Fig. 18 are shown the detail drawings of a reinforced-concrete bridge having a clear span of 62 feet 8¼ inches. The arch portion of the bridge is laid out on three centers, and the bridge is built on a skew; for this reason, the duplication of the structure in stonework would be very costly. In (*a*) is shown a half plan of the bridge; in (*b*), a cross-section, on the line *A A*; and in (*c*), the detailed design of the wells, or shafts, arranged back of the buttress walls in order that access may be had to pipes, etc. A section through the buttresses, which tend to brace the side walls of the bridge, is shown in (*d*).

19. Referring to the section shown in (*b*), it will be seen that the side, or buttress, walls of the arch construction are 6 feet thick, with an increase of a foot in thickness at the bottom. These walls are reinforced with ⅞-inch vertical square bars placed 2 feet 8 inches from center to center. The bars are placed near each face of the wall and project up into the spring of the arch, lapping with the reinforcing bars of the arch construction. There are no horizontal rods extending lengthwise of the buttress walls, though it would be good practice to supply some to take care of the shrinkage in the concrete. The arch ring, as will be observed from the section, is very heavy and deep toward the abutments and

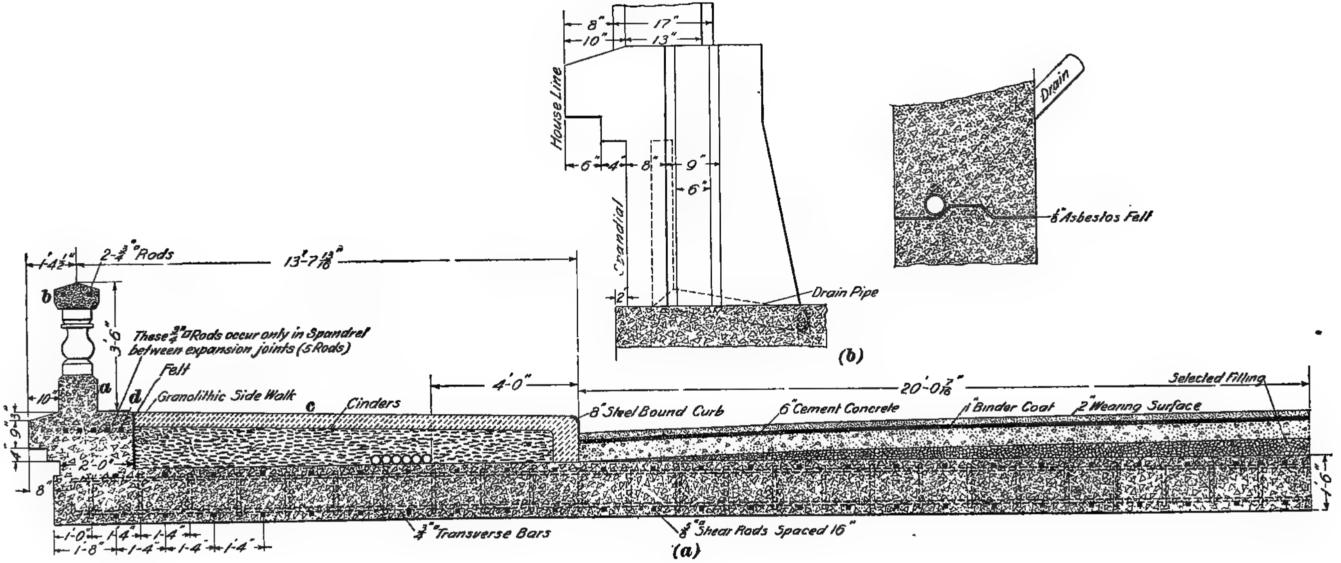

House Line
8"
17"
10"
13"
6"
4"
8"
9"
6"
Spandrel
2
Drain Pipe
(b)
Drain
⅛" Asbestos Felt
1'-4½"
2-¾" Rods
13'-7 13/16"
b
3'-6"
These ¾" □ Rods occur only in Spandrel between expansion joints (5 Rods)
Felt
a
d
Granolithic Side Walk
c
Cinders
4'-0"
20'-0 7/16"
Selected Filling
8" Steel Bound Curb
6" Cement Concrete
1" Binder Coat
2" Wearing Surface
10"
3"
6"
4"
2'-0"
8"
1'-6"
1'-0"
1'-4"
1'-4"
1'-8"
1'-4"
1'-4"
1'-4"
¾" Transverse Bars
⅝" Shear Rods Spaced 16"
(a)

FIG. 19

thin at the top of the arch. It is reinforced throughout with ⅞-inch square bars placed 3 inches from the upper and lower surfaces of the arch ring. Besides these rods there are ¾-inch square bars running across the arch. These rods are spaced as shown in the section, are connected by means of shear rods provided in one length from the crown of the arch to the abutments, and are bent so as to pass outside the transverse rods. These shear rods are placed 16 inches from center to center, and where the two shear rods required to span from buttress to buttress meet at the center of the arch, they are lapped 5 feet.

20. The section shown in (*b*), as well as the detail shown in (*d*), illustrates clearly the manner in which the vertical rods of the buttress walls and the rods of the arch ring interlace and lap. It will be observed from the section shown in (*b*) that the buttresses are arranged to be constructed separately from the arch ring. They are formed with a shoulder, so that when the concrete of the arch is joined with them, there will be no tendency for the arch to slide or move on them.

In nearly all concrete bridges supporting roadways, a spandrel is formed along both side faces of the arch, as shown at *b*. This is done in order to hold the road construction in place and to provide a curb for the support of a balustrade or railing. It is also customary, in order to prevent the percolation of surface water through the crown of the arch, to waterproof the top, or extrados, of the arch construction. In the design shown, this waterproofing is omitted, but the expansion joints in the coping along the side faces of the arch are made waterproof with three coats of *Hydrex felt*, which is a patented waterproof material in the nature of felt saturated with pitch. The intrados, or inside surface of the arch, is finished with 1-inch granolithic face, or cement finish, and this finish is carried down the surfaces of the buttress walls.

21. To explain more clearly the several details of the road construction and the reinforcement of the arch at the crown, reference is made to Fig. 19 (*a*), which shows a half section through the crown of the arch and the roadway it supports.

As will be observed, the thickness at the crown of the arch is 1 foot 6 inches. The finish of the coping is shown at *a*, and the balustrade protecting the sidewalks, at *b*. The roadway is graded with selected filling, upon which 6 inches of cement concrete is placed, and upon the top of this concrete is placed a 1-inch binder coat and a 2-inch wearing surface. The pavement is graded to the proper level with cinders and is finished with a granolithic sidewalk, as shown at *c*. It will be noticed that the rail of the balustrade is reinforced with two ¾-inch square rods, and that five ¾-inch square rods run lengthwise of the spandrel section, as shown at *d*.

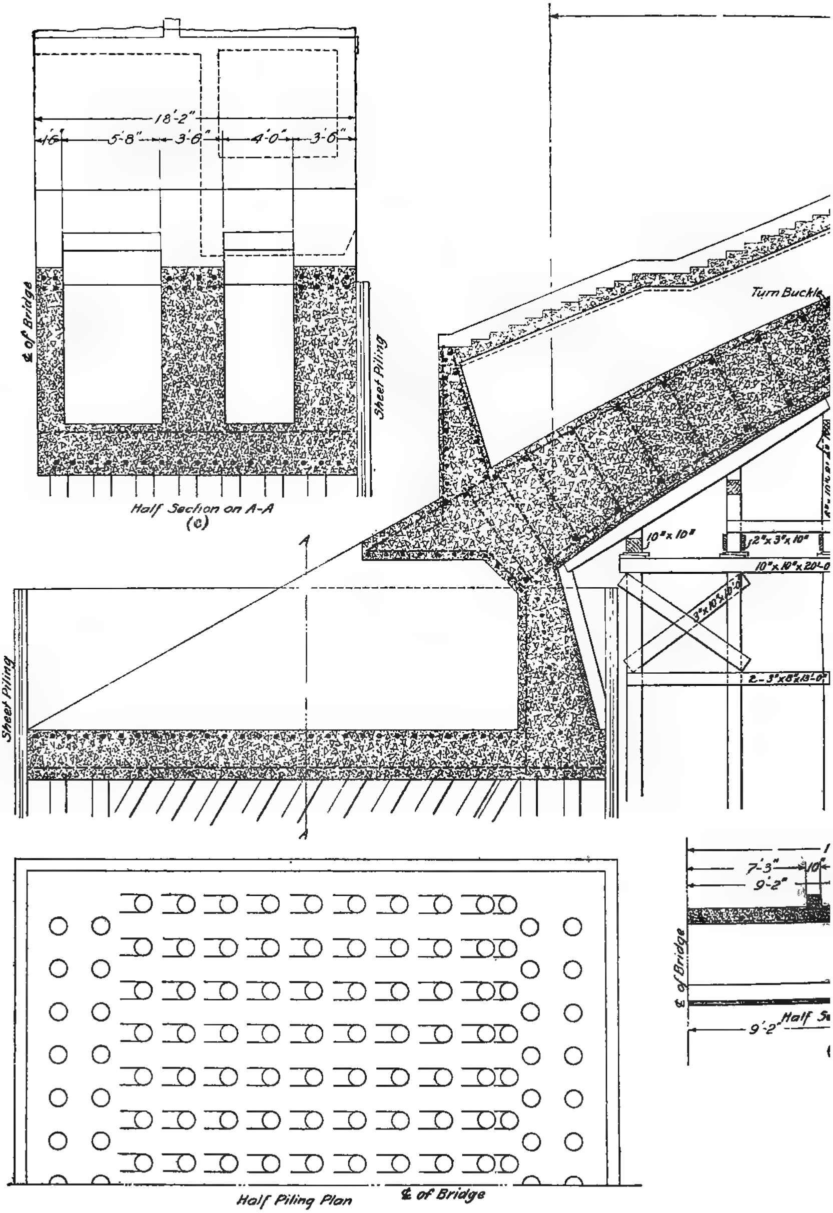
In Fig. 19 (*b*) is shown the method of forming the expansion joint in the coping, or spandrel, as well as the pipe used to drain the coping and the upper portion of the sidewalk in position.

The other details of the construction and reinforcement are indicated on the drawing, and may be used for reference in the design and construction of work of this character.

22. Bridge of Large Span.—In Fig. 20 is shown a large segmental arch constructed of reinforced concrete. This arch was built for the purpose of connecting the government piers at the Jamestown Exposition. The illustration shows in detail the construction of the reinforced-concrete arch, its spandrels and abutments, and likewise the method of framing the timber centering for the support and construction of the arch.

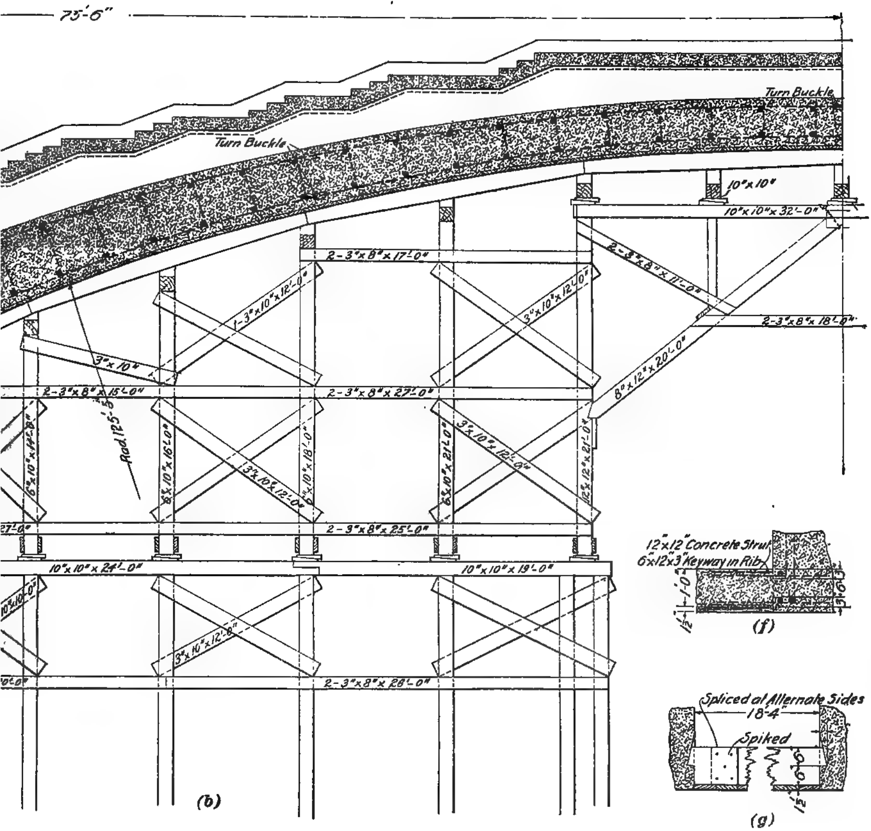
According to the dimensions given in the figure, the span of the arch from the intersection of the intrados with the line of the abutment is 151 feet, and the arch is built on a radius of 125 feet 6 inches. As the ground upon which the construction was placed is comparatively poor, the abutments were supported entirely upon timber piling driven as indicated on the plan shown in (*a*).

23. An investigation of the half section through the arch at (*d*) will show that the depth of the arch ring at the center is 2 feet 10 inches although the depth of the ring at the abutment is about 7 feet. From the section shown in (*c*), which



Half Section on A-A
(c)

Half Piling Plan
(a)


75'-6"
Turn Buckle
Turn Buckle
10"x10"
10"x10"x32'-0"
2-3"x8"x17'-0"
2-3"x8"x11'-0"
2-3"x8"x18'-0"
1-3"x10"x12'-0"
3"x10"x12'-0"
3"x10"
8"x12"x20'-0"
2-3"x8"x15'-0"
2-3"x8"x27'-0"
Rad. 125'-3"
6"x10"x14'-8"
6"x10"x16'-0"
3"x10"x12'-0"
6"x10"x18'-0"
6"x10"x21'-0"
3"x10"x12'-0"
12"x12"x21'-0"
27'-0"
2-3"x8"x25'-0"
10"x10"x24'-0"
10"x10"x19'-0"
10"x10'-0"
3"x10"x12'-0"
2-3"x8"x28'-0"

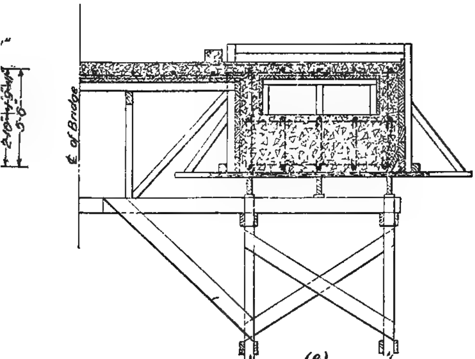
(b)

12"x12" Concrete Strut
6"x12"x3" Keyway in Rib
1'-0"
3'-6"
12"

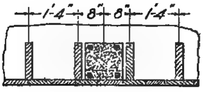
(f)

Spliced at Alternate Sides
18'-4"
Spiked
1'-0"
12"

(g)


2'-10"
5'-6"
℄ of Bridge

(e)


1'-4"
8"
8"
1'-4"

(h)

is taken along the line *A A* through the abutment, it will be seen that the abutment is not a solid mass of concrete, but is in the nature of a reinforced-concrete footing and abutment wall strengthened with reinforced-concrete ribs. By this construction the strength of the abutment is little impaired and the quantity of concrete required is greatly reduced. As will be observed from the section of the abutment shown in (*b*), some of the piling is driven obliquely, so as to resist the resultant thrust from the arch and thus accomplish what the abutment would fail to do through its lack of mass and weight.

From the half section shown in (*d*), it will also be noticed that the arch ring is not of solid concrete, but consists of two parallel rings 9 feet in width and connected with a reinforced-concrete slab. This slab construction is embodied with a series of steps, as shown in (*b*), and is supported from the rings by spandrel walls of reinforced concrete. Each rib of the arch is reinforced with ten 1½-inch bars connected with either sleeve nuts or turnbuckles. Five of these bars are placed in the bottom of the ring and five in the top, and they are well tied together with bars in the nature of loop ties.

Another half section through the arch ring is shown in (*e*), which also indicates a cross-section through the centering and its supports, and illustrates the method by which were constructed the spandrel walls that support the steps forming the travelway of the bridge.

From the section shown in (*b*), it will be seen that both the abutment and the footings are strongly reinforced with ¾-inch bars. These are arranged 12 inches from center to center in the footing. Similar rods or bars are run lengthwise throughout the abutment wall and thus provide the necessary resistance against shrinkage cracks.

Other details of the construction are shown in Fig. 20 (*f*), (*g*), and (*h*). In (*g*) and (*h*) is illustrated the method of lagging the soffit of the arch with wood. This makes a neat finish and covers up the rough work underneath the step construction. Such construction is unusual, however, and a soffit of cement construction would be more common.

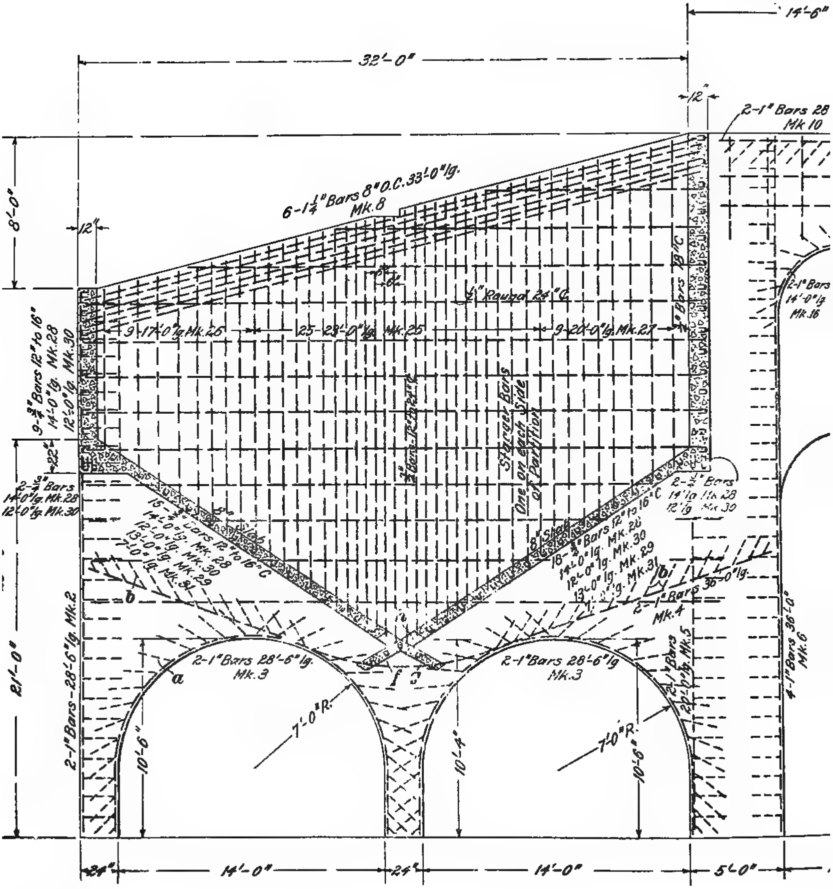
24. The bridge just described is an excellent example of economical construction, and shows what can be accomplished with reinforced concrete when used for bridges supporting walkways or roadways. There are many other types of reinforced-concrete bridges for this purpose, with spans ranging from 75 to 150 feet. The material lends itself admirably to architectural treatment, and by employing graceful soffit curves and neat balustrade and spandrel effects, as well as a good surface finish, excellent architectural results will be obtained.

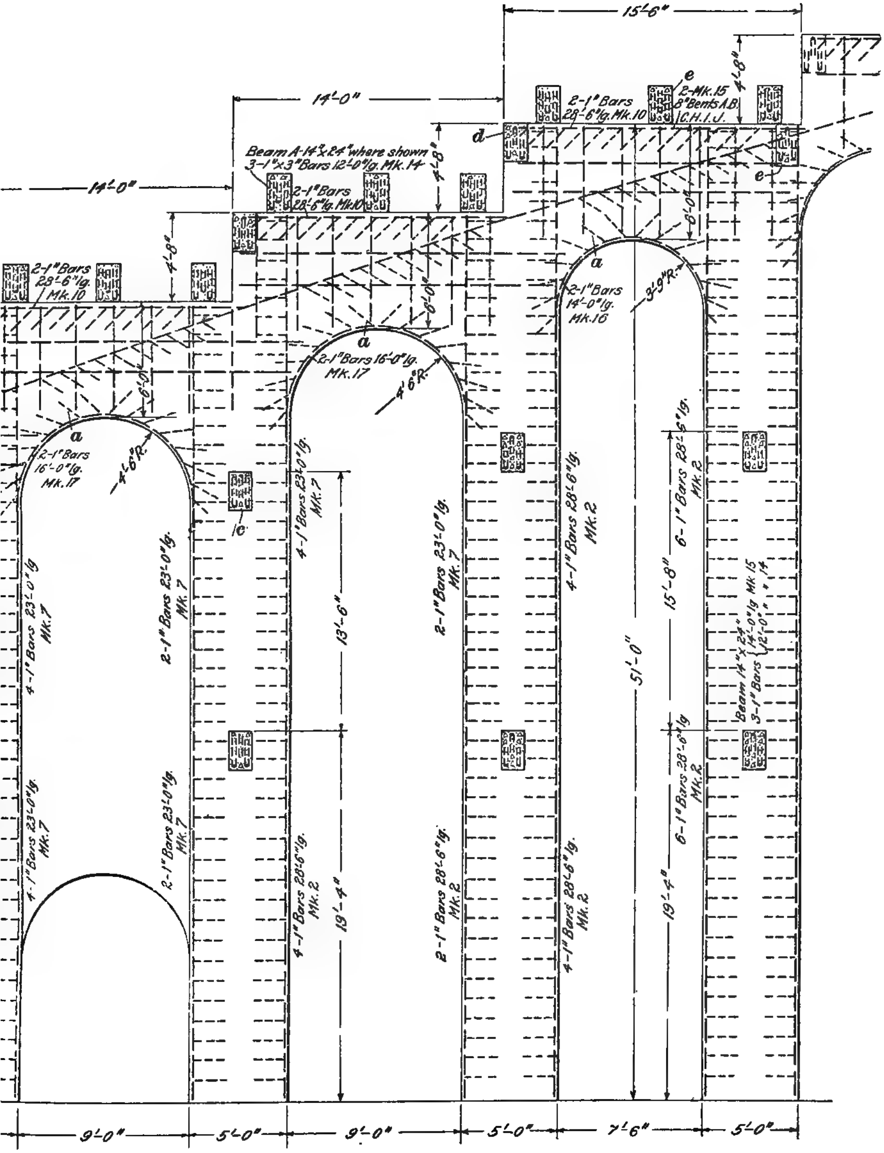
COAL BREAKER OF REINFORCED CONCRETE

25. Reinforced concrete has been employed in the construction of coal breakers. Ordinarily, these structures are not of such a permanent character as to make it economical or practical to use reinforced concrete throughout their construction. On the other hand, they are subject to great fire risk, and their destruction entails considerable loss in a mining operation. Besides, on account of the continual moisture incident to the preparation of coal, wooden structures rot quickly.

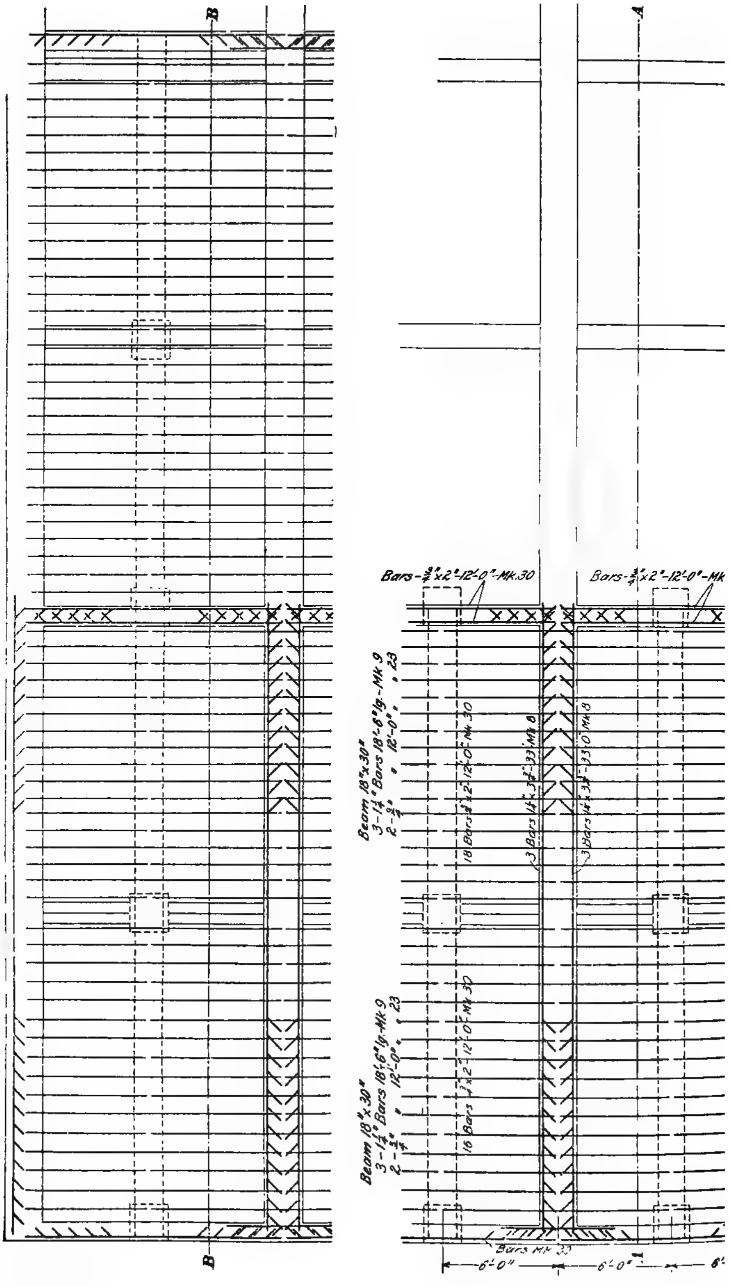
26. The following illustrations show the details of reinforced concrete used in a coal breaker of considerable size near Pottsville, Pennsylvania. In Fig. 21 is shown a detailed section through the bins, or coal pockets, of the breaker, as well as the construction and reinforcement of the supporting walls over the driveways and tracks. According to the dimensions given in this figure, the work is of considerable proportions; this is necessary, however, because such structures must be strongly built and reinforced in order to carry the great loads due to the coal and the weight of the structure above.

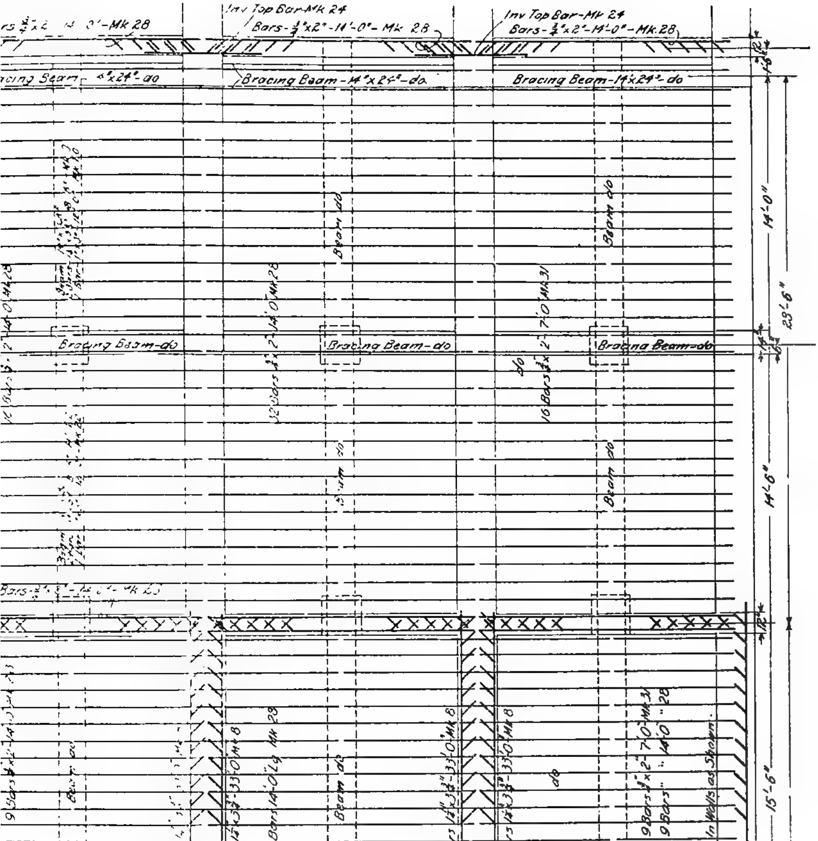
From the plan shown in Fig. 22, the general outline and construction of the breaker may be seen. The walls of the breaker are constructed with arched openings, as shown in Fig. 21, the intermediate supports of the bins and hoppers being rectangular columns, which are shown in elevation in


14'-6"
32'-0"
12"
2-1" Bars 28
Mk 10
6-1¼" Bars 8" O.C. 33'-0" lg.
Mk. 8
8'-0"
12"
½" Round 24" C.
¾" Bars 28" C
9-17'-0" lg. Mk. 26
25-23'-0" lg. Mk. 25
9-20'-0" lg. Mk. 27
9-¾" Bars 12" to 16"
14'-0" lg. Mk. 28
12'-0" lg. Mk. 30
22"
2-¾" Bars
14'-0" lg. Mk. 28
12'-0" lg. Mk. 30
Stirrup Bars
One on each Side
of Partition
2-1" Bars
14'-0" lg.
Mk. 16
2-¾" Bars
14' lg. Mk. 28
12' lg. Mk. 30
15-¾" Bars 12" to 16" C
14'-0" lg. Mk. 28
12'-0" lg. Mk. 30
13'-0" lg. Mk. 29
7'-0" lg. Mk. 31
8" Slab
8" Slab
18-¾" Bars 12" to 16" C
14'-0" lg. Mk. 28
12'-0" lg. Mk. 30
13'-0" lg. Mk. 29
7'-0" lg. Mk. 31
b
b
2-1" Bars 36'-0" lg.
Mk. 4
2-1" Bars 28'-6" lg.
Mk. 3
2-1" Bars 28'-6" lg.
Mk. 3
a
f
c
2-1" Bars 20'-0" lg. Mk. 5
4-1" Bars 36'-0"
Mk. 6
2-1" Bars-28'-6" lg. Mk. 2
21'-0"
10'-6"
7'-0" R.
10'-4"
7'-0" R.
10'-6"
24"
14'-0"
24"
14'-0"
5'-0"


15'-6"
14'-0"
14'-0"
4'-8"
4'-8"
4'-8"
e
2-1" Bars 28'-6"lg. Mk.10
2-Mk.15 8" Bents A.B. C.H.I.J.
d
Beam A-14"x24" where shown
3-1"x3" Bars 12'-0"lg. Mk.14
2-1" Bars 28'-6"lg. Mk.10
2-1" Bars 28'-6"lg. Mk.10
e
6'-0"
6'-0"
6'-0"
a
a
a
2-1" Bars 14'-0"lg. Mk.16
3'-9"R.
2-1" Bars 16'-0"lg. Mk.17
4'-6"R.
2-1" Bars 16'-0"lg. Mk.17
4'-6"R.
c
4-1" Bars 23'-0"lg. Mk.7
2-1" Bars 23'-0"lg. Mk.7
4-1" Bars 23'-0"lg. Mk.7
2-1" Bars 23'-0"lg. Mk.7
4-1" Bars 23'-0"lg. Mk.7
2-1" Bars 23'-0"lg. Mk.7
13'-6"
4-1" Bars 28'-6"lg. Mk.2
6-1" Bars 28'-6"lg. Mk.2
15'-8"
51'-0"
Beam 14"x24" 3-1" Bars {14'-0"lg Mk.15 12'-0" " " 14
6-1" Bars 28'-6"lg. Mk.2
4-1" Bars 28'-6"lg. Mk.2
2-1" Bars 28'-6"lg. Mk.2
4-1" Bars 28'-6"lg. Mk.2
19'-4"
19'-4"
9'-0"
5'-0"
9'-0"
5'-0"
7'-6"
5'-0"

tion A-A


B
A
Bars-3/4"x2"-12'-0"-Mk.30
Bars-3/4"x2"-12'-0"-Mk
Beam 18"x30"
3-1 1/4" Bars 18'-6" lg.-Mk 9
2-3/4" " 12'-0" " " 23
18 Bars 3/4"x2"-12'-0"-Mk 30
3 Bars 1/4"x3 1/2"-33'-0" Mk 8
3 Bars 1/4"x3 1/2"-33'-0" Mk 8
Beam 18"x30"
3-1 1/4" Bars 18'-6" lg.-Mk 9
2-3/4" " 12'-0" " " 23
16 Bars 3/4"x2"-12'-0"-Mk 30
Bars Mk 23
6'-0"
6'-0"
6'
B


Inv Top Bar-Mk 24
Bars-3/4"x2"-14'-0"-Mk 28
Inv Top Bar-Mk 24
Bars-3/4"x2"-14'-0"-Mk.28
Bracing Beam-14"x24"-do
Bracing Beam-14"x24"-do
Bracing Beam-do
Bracing Beam-do
Bracing Beam-do
Beam do
Beam do
Beam do
Beam do
12 Bars 3/4"x2"-14'-0"-Mk 28
16 Bars 3/4"x2"-7'-0"-Mk 31
do
18 Bars 14'-0" Lg Mk 29
3 Bars 1/4"x3 3/4"-33'-0"-Mk 8
3 Bars 1/4"x3 3/4"-33'-0"-Mk 8
3 Bars 1/4"x3 3/4"-33'-0"-Mk 8
9 Bars 3/4"x2"-7'-0"-Mk 31
9 Bars " " 14'-0" " 28
3 Bars 1/4"x3 3/4"-33'-0"-Mk 8 In Walls as Shown
16 Bars 3/4"x2"-14'-0"-Mk 28
8 Bars 3/4"x2"-7'-0"-Mk 31
0 " " 14'-0" " 28
Note - For exact Location of Bars for Slab Reinforcement see Section A-A
b
b
a
c
c
14'-0"
14'-6"
29'-6"
15'-6"
17'-0"
32'-6"
12"
12"
12"
7'-0"
7'-0"
7'-0"
7'-0"
7'-0"
7'-6"

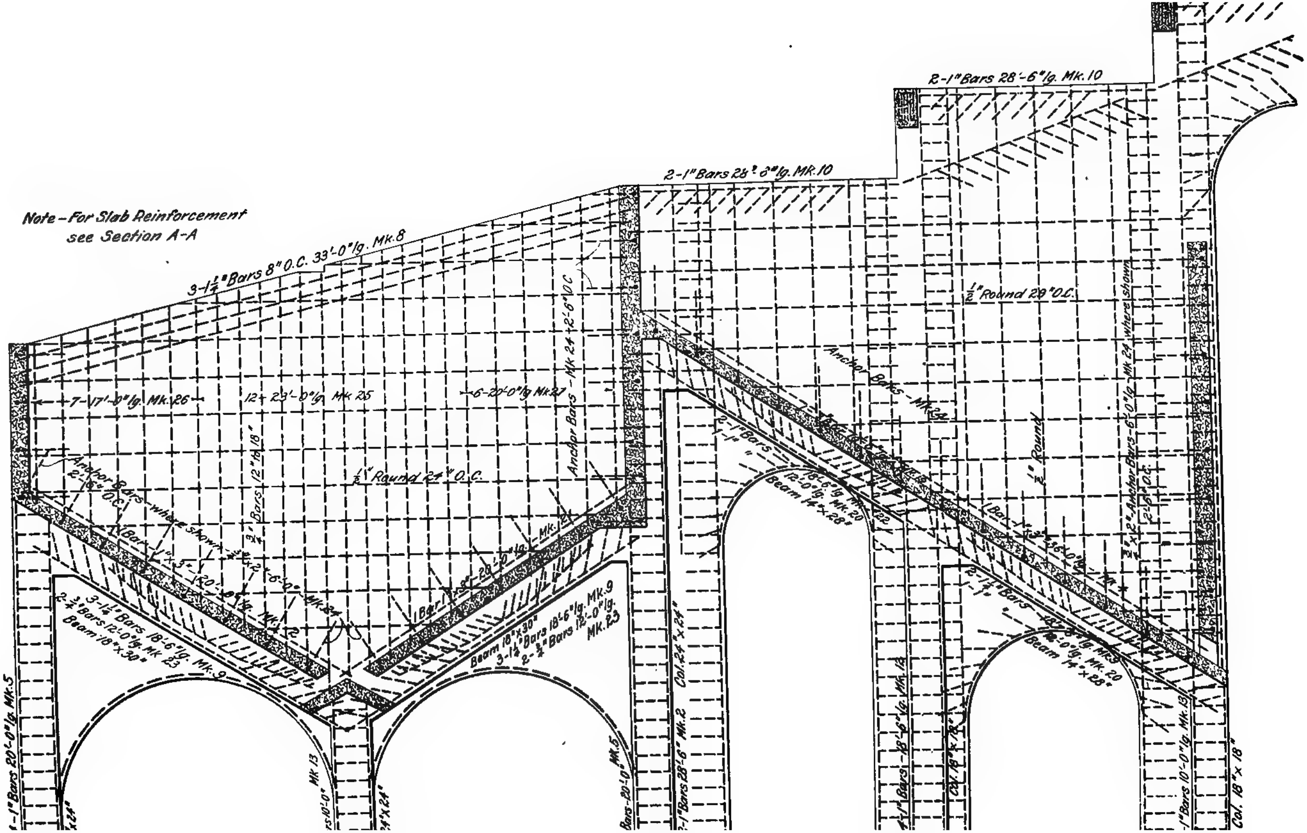
the detail section, Fig. 23. The reinforcement of this breaker consists generally of Kahn truss bars. The facility with which these bars can be used for reinforcement over the arched openings and elsewhere in the structure will be apparent on referring to *a*, Fig. 21. The method of using the bars shown at *b* is also unique; it illustrates some of the advantages of a reinforcing bar that has sheared stirrups. All the cross-beams to support the machinery and the upper portion of the structure, as at *c*, *d*, and *e*, are reinforced with truss bars, a shear bar being in all instances placed at the top of the beams over their bearing on the piers or side walls.

27. The sides of the bins are heavily reinforced to resist the lateral pressure of the coal contained in them. This reinforcement likewise consists of Kahn truss bars, although here the stirrups are not of great length. One of the peculiar features of the design is in the division wall between the bins. This wall must be reinforced near both surfaces, as one bin may be full of coal while the other is empty, and the bending stresses may therefore be reversed. Another peculiarity of the design is the manner in which the bottom slabs of the hopper, as at *f* and *g*, are extended. These extensions of the slabs act as loading chutes for the coal coming from the hopper outlets, or openings, at *h*.

Most of the vertical bars used as shrinkage bars are of ½-inch rounds, because this type of bar is the most economical to use for such work.

The various marks on the drawings, such as *Mk.* with a number following, are simply reference marks. These marks enable the reinforcement to be followed on the several drawings, or plans. The bars are also tagged with these marks, so that the erector will know where they belong.

28. The plan of the structure, Fig. 22, besides showing the location of the cross-walls, as at *a*, shows the location of the intermediate column supports, as indicated at *b*. From the plan of the cross-walls, it will be seen that the reinforcing bars are placed near each face of the wall, with the sheared stirrups projecting inwards. A study of the exterior wall of

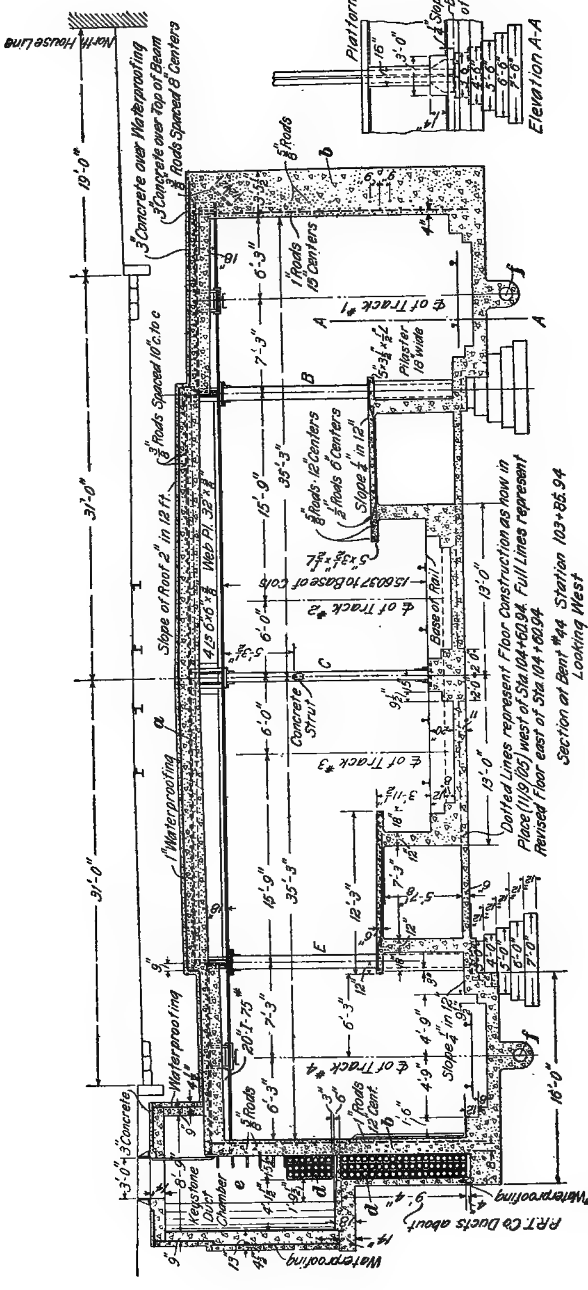

2-1" Bars 28'-6" lg. Mk.10
2-1" Bars 28'-6" lg. Mk.10
3-1¼" Bars 8" O.C. 33'-0" lg. Mk.8
7-17'-0" lg. Mk.26
12-23'-0" lg. Mk 25
6-20'-0" lg. Mk.27
Anchor Bars - Mk 24 - 2'-6" O.C.
½" Round 24" O.C.
¾" Bars 12" to 18"
Anchor Bars – where shown 2'-6" O.C.
1 Bar – 1" × 3" – 20'-0" lg. Mk.12
¾" × 2" – 6'-0" Mk.24
1 Bar 1" × 3" – 20'-0" lg. – Mk.12
3-1¼" Bars 18'-6" lg. Mk.9
2-¾" Bars 12'-0" lg. Mk 23
Beam 18" × 30"
Beam 18" × 30"
3-1¼" Bars 18'-6" lg. Mk.9
2-¾" Bars 12'-0" lg. Mk.23
1-1" Bars 20'-0" lg. Mk.5
Mk 13
Mk.5
Col. 24" × 24"
2-1" Bars 28'-6" Mk.2
½" Round 29" O.C.
Anchor Bars – Mk.24
2-1" Bars 18'-6" lg.
1-1" " 12'-0" lg. Mk.20
Beam 14" × 28"
Mk.22
½" Round
1 Bar – 1" × 3" – 36'-0" lg. – Mk.
¾" × 2" Anchor Bars – 6'-0" lg. – Mk 24 where shown
2'-6" O.C.
2-1¼" Bars 18'-6" lg. Mk.3
1-1" " 12'-0" lg. Mk.20
Beam 14" × 28"
1" Bars – 18'-6" lg. Mk.18
Col. 18" × 18"
1" Bars 10'-0" lg. Mk.13
Col. 18" × 18"

Note – For Slab Reinforcement see Section A-A the bin, as at *c*, will show the method of using the shear bar at the junction, or bearing, of the sides of the slabs of the bin with the cross-wall. By using these shear bars, the structure is greatly strengthened at these junctions, and the benefit of a continuous beam is obtained. The other details of the construction are clearly shown in the several figures, as is also the application of steel reinforcement and concrete work in the construction of such structures as breakers and storage bins.

REINFORCED-CONCRETE SUBWAY WORK

29. Reinforced concrete has been used extensively in the construction of subways in several of the principal cities. In Fig. 24 is shown a cross-section of the double-track subway at one of the stations in Philadelphia. In this instance, the roof of the subway is supported on structural-steel plate girders that carry a heavy reinforced-concrete slab, as shown at *a*. These girders are supported at both sides by structural-steel columns, and intermediately by structural-steel columns braced with reinforced-concrete struts that extend between them. The side walls are of the section indicated at *b*. They are reinforced vertically with 1-inch square twisted bars placed 12 inches on centers, and horizontally with ⅝-inch shrinkage rods or bars. The floor of the subway is of plain concrete. The station platforms are constructed upon supporting walls 12 inches in thickness and consist of a 6-inch slab of concrete reinforced with steel rods or bars running across and lengthwise of the construction. These rods or bars are of the size and spacing indicated. The edges of the platforms are reinforced with steel angles.

As will be observed from the illustration, the top of the subway is some distance below the surface of the street and pavement; also, this top, or roof, is graded toward both sides of the tunnel in order to drain any water that percolates through the soil. The top of the concrete slab covering is waterproofed, the waterproofing being 1 inch in thickness and protected over the top with 3 inches of concrete. The con-



Section at Bent #44 Station 103+86.94
Looking West

FIG. 24

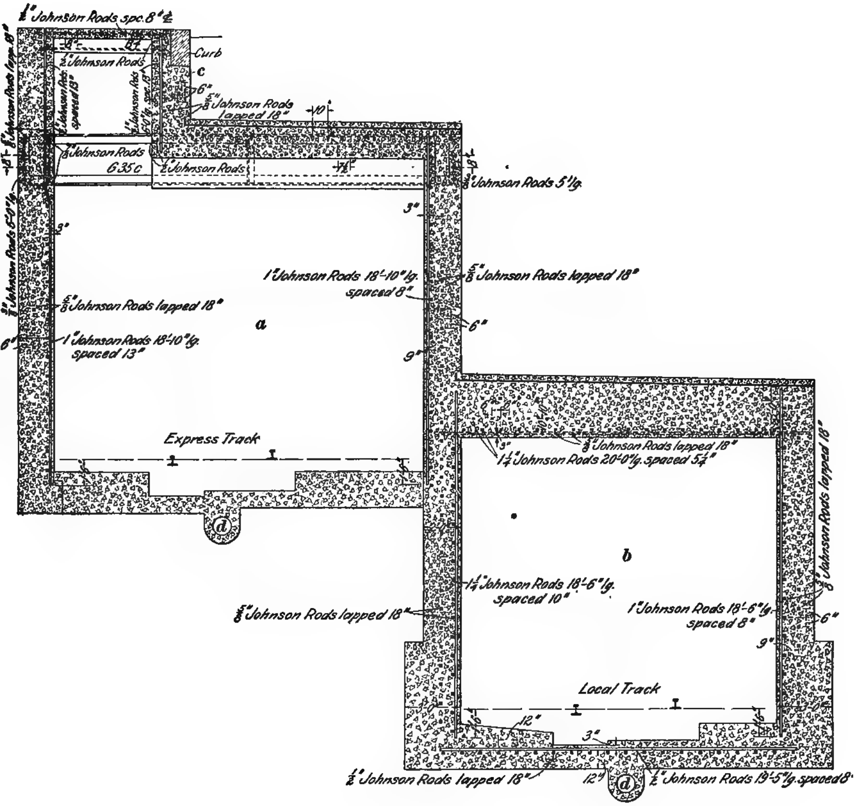
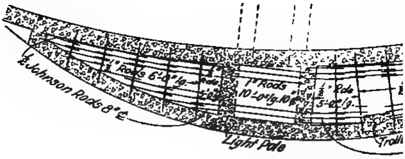

1/2" Johnson Rods spc. 8" ℄
Curb
c
1/2" Johnson Rods
1/2" Johnson Rods spaced 18"
1/2" Johnson Rods 6'-0" lg. spc. 18"
6"
5/8" Johnson Rods lapped 18"
10"
5/8" Johnson Rods lapped 18"
5/8" Johnson Rods
G 35 c
1/2" Johnson Rods
7 1/2"
18"
5/8" Johnson Rods 5' lg.
3/4" Johnson Rods 5'-0" lg.
3"
3"
1" Johnson Rods 18'-10" lg. spaced 8"
5/8" Johnson Rods lapped 18"
5/8" Johnson Rods lapped 18"
a
6"
6"
1" Johnson Rods 18'-10" lg. spaced 13"
9"
Express Track
d
3"
5/8" Johnson Rods lapped 18"
1 1/4" Johnson Rods 20'-0" lg. spaced 5 1/4"
5/8" Johnson Rods lapped 18"
b
1 1/4" Johnson Rods 18'-6" lg. spaced 10"
5/8" Johnson Rods lapped 18"
1" Johnson Rods 18'-6" lg. spaced 8"
6"
9"
Local Track
12"
3"
1/2" Johnson Rods lapped 18"
12"
d
1/2" Johnson Rods 19'-5" lg. spaced 8"

FIG. 25

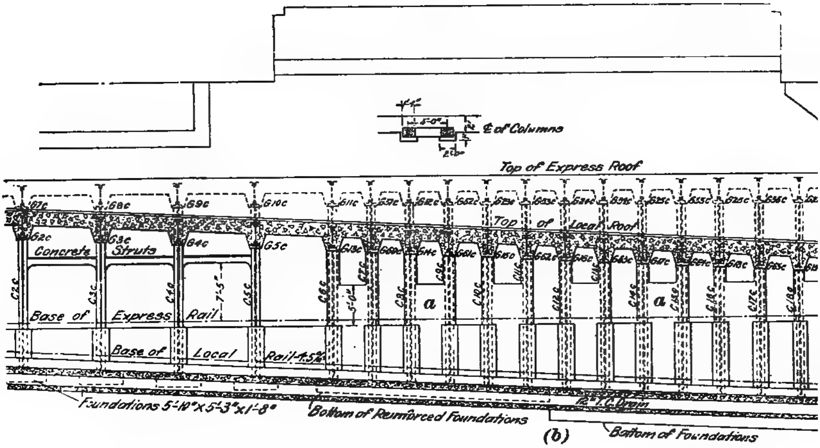
struction shown at *d* indicates terra-cotta pipes, or conduits, in which electric wires are run. There is also provided, as shown at *e*, a duct chamber with a manhole entrance from the street. By means of this chamber access may be had to the upper tier of conduits, which are used for telephone wires. At *f* are shown drains to drain off any water that may be on the floor of the tunnel.

30. In Fig. 25 is shown the construction of the two reinforced-concrete tunnels forming the Market Street subway in Philadelphia. The upper tunnel *a* runs out upon grade, and the lower one *b* slopes down below grade. The lower tunnel, which is used for the local track, is built entirely of reinforced concrete. The bottom, sides, and top are reinforced with transverse and longitudinal bars, pitched as shown in the section, the deformed bar known as the Johnson bar being employed. The upper tunnel, which is used for the express track, is roofed by using built-up steel girders covered with concrete and spanning the tunnel and supporting a reinforced-concrete slab between them, a wall being built up to carry the pavement curb, as shown at *c*. It will be observed in this illustration, as well as in the preceding one, that the space beneath the track is depressed, and there is provided an encased drain *d* to carry away any water that might accumulate in the tunnel. As will be noted, the steel girders in the roof of the tunnel *a* are tied into the concrete with short Johnson bars put through their top flanges.

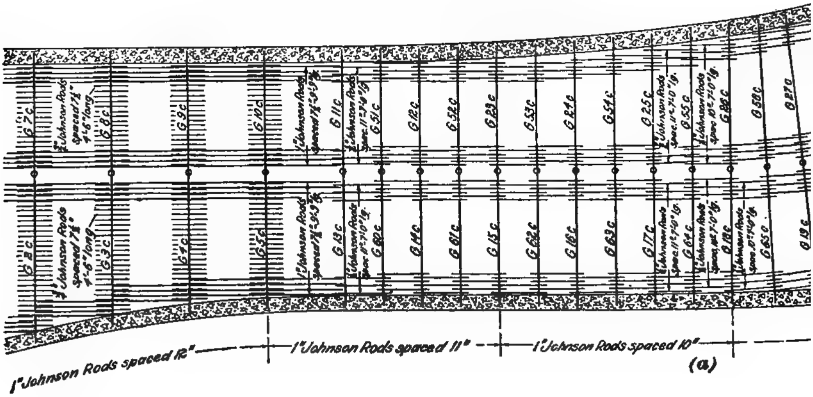
31. The section just described will be better understood by referring to Fig. 26, which shows a plan and a part elevation or developed section of the subway construction around the Philadelphia city hall. The section shown in Fig. 25 is taken on the girder line *G 35 C*, which is shown in Fig. 26 (*a*). The developed section shown in Fig. 26 (*b*) clearly illustrates the method of supporting the steel girders that carry the reinforced-concrete slab. In many instances they are encased with concrete, and between every other one a slab of concrete is monolithic with the casing of the steel column, as shown at *a*. The developed section (*b*) also shows the manner of

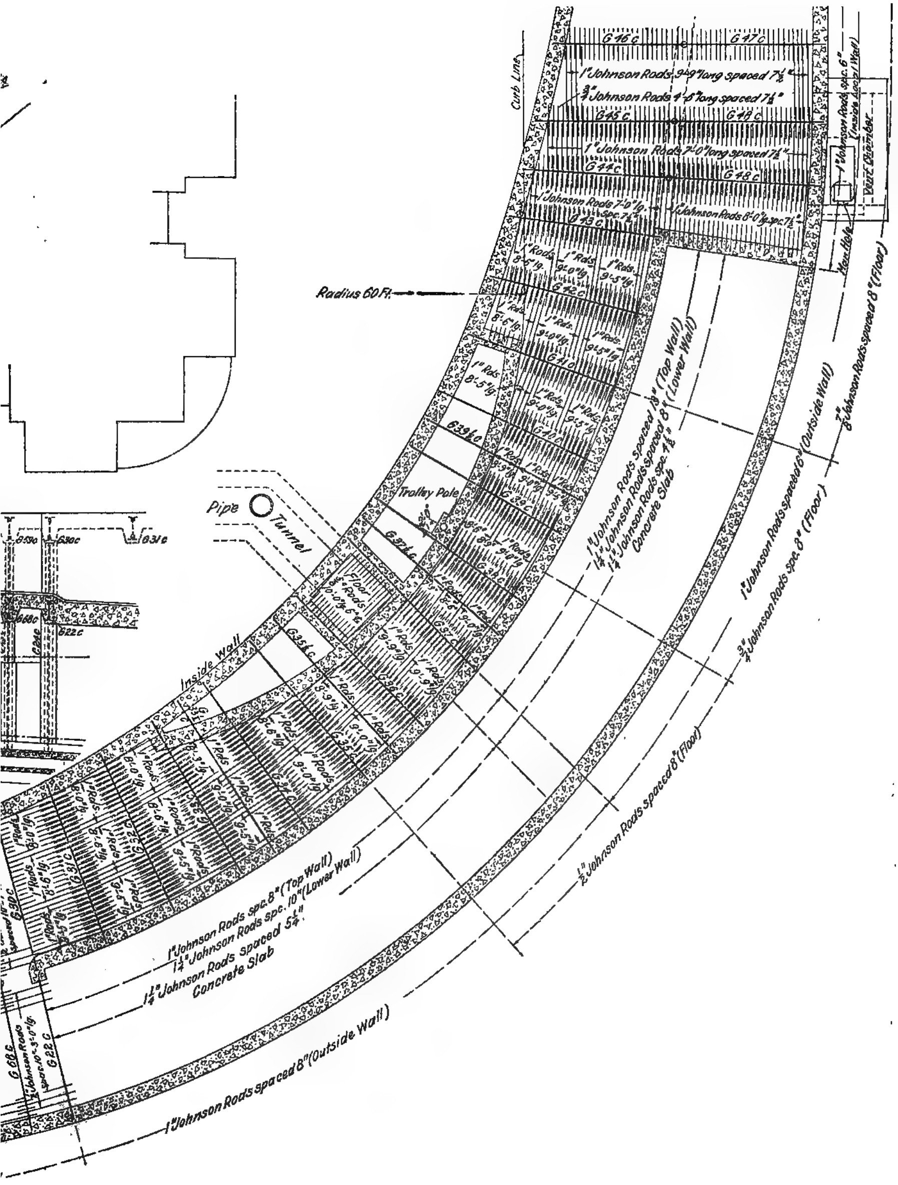


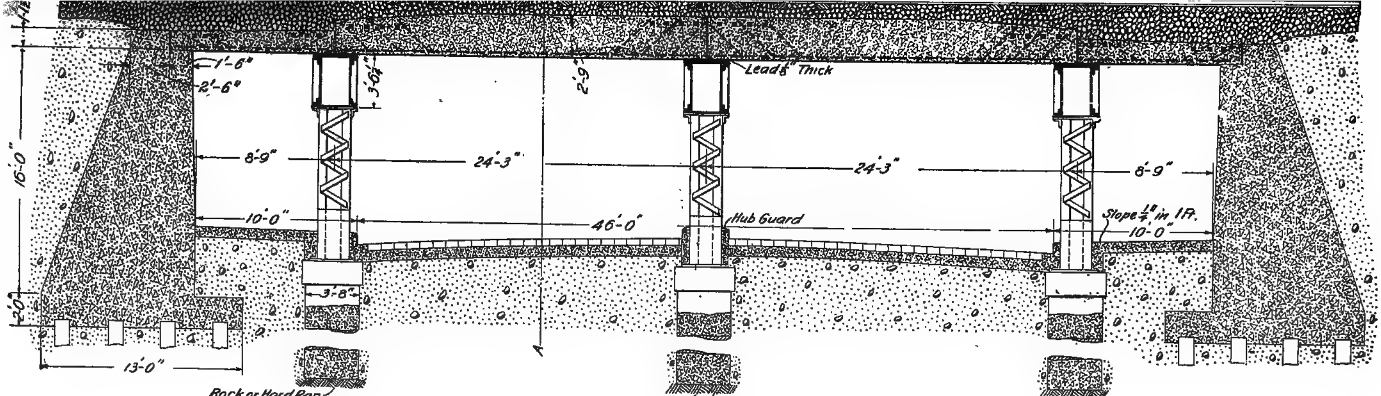
Roof Plan S.E.Corne



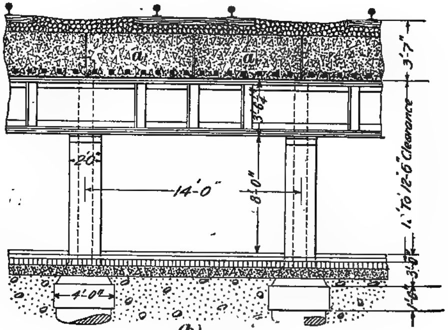
Developed Section
on ℄ of Local Track "G2c to G22c"




Curb Line
G 46 c
G 47 c
1" Johnson Rods 9'-9" long spaced 7½"
¾" Johnson Rods 4'-6" long spaced 7½"
G 45 c
G 49 c
1" Johnson Rods 7'-0" long spaced 7½"
G 44 c
G 48 c
1" Johnson Rods 7'-0" lg. spc. 7½"
1" Johnson Rods 8'-0" lg. spc. 7½"
G 43 c
1" Johnson Rods spc. 6" (Inside Loop Wall)
Vent Chamber
Man Hole
Radius 60 Ft.
1" Rods 8'-5" lg.
1" Rods 9'-0" lg.
1" Rods 9'-5" lg.
G 42 c
G 41 c
G 40 c
G 39 c
G 39½ c
G 38 c
G 37½ c
G 37 c
G 36 c
G 35½ c
G 35 c
G 34 c
G 33 c
G 32 c
G 31 c
G 30 c
G 22 c
G 68 c
Trolley Pole
Pipe
Tunnel
Floor 1" Rods 10'-0" lg. 6" c.
Inside Wall
1" Johnson Rods spaced 18" (Top Wall)
1¼" Johnson Rods spaced 8" (Lower Wall)
1¼" Johnson Rods spc. 4½"
Concrete Slab
1" Johnson Rods spaced 6" (Outside Wall)
⅜" Johnson Rods spaced 8" (Floor)
1" Johnson Rods spc. 8" (Floor)
¾" Johnson Rods spc. 8" (Floor)
½" Johnson Rods spaced 8" (Floor)
1" Johnson Rods spc. 8" (Top Wall)
1¼" Johnson Rods spc. 10" (Lower Wall)
1¼" Johnson Rods spaced 5¼"
Concrete Slab
1" Johnson Rods spaced 8" (Outside Wall)
1" Johnson Rods spac. 10" 3'-0" lg.
G 59 c
G 30 c
G 31 c
G 68 c
G 22 c



(a)
Cross Section of Subway



(b)
Section on Line A-B

FIG. 27

encasing the steel beams or girders. Other details of the construction are noted or shown on the plan, and they should be studied as a precedent for work of this character.

REINFORCED-CONCRETE ELEVATED-RAILROAD WORK

32. Reinforced concrete is extensively used in elevated-railway work. For this purpose, it has some advantages over steel construction, in that the maintenance charges are reduced, the concrete requiring no painting and very little inspection.

33. An example of this type of construction is shown in Fig. 27 (*a*), which illustrates a C. B. & Q. R. R. track elevation over a city street. In this instance, the abutment walls consist of heavy concrete supported on piles, and the main intermediate supports are structural-steel columns that carry structural-steel girders of box section. The support for the trackwork consists of solid slabs of reinforced concrete, those over the large spans of 24 feet 3 inches being 2 feet 9 inches in thickness and heavily reinforced. The smaller spans between the structural-steel columns nearest the abutments and the abutment walls, which are 8 feet 9 inches, are not so thick, however, on account of the reduced bending stresses.

From the section shown in (*b*), it will be seen that the slab construction is in sections, being entirely cut through at intervals, as shown at *a*. This construction does not weaken the slab materially, and also facilitates the construction of the work.

In order to take up all inequalities between the slab construction and the steelwork, a lead packing piece ⅛-inch thick is placed between the bearing of the concrete and the steel box girder. This sheet lead also protects the top of the steel girders from moisture and possible corrosion due to dampness coming through the concrete. This type of construction is very economical in labor on form work, and possesses an advantage in that the slab work over the street does not

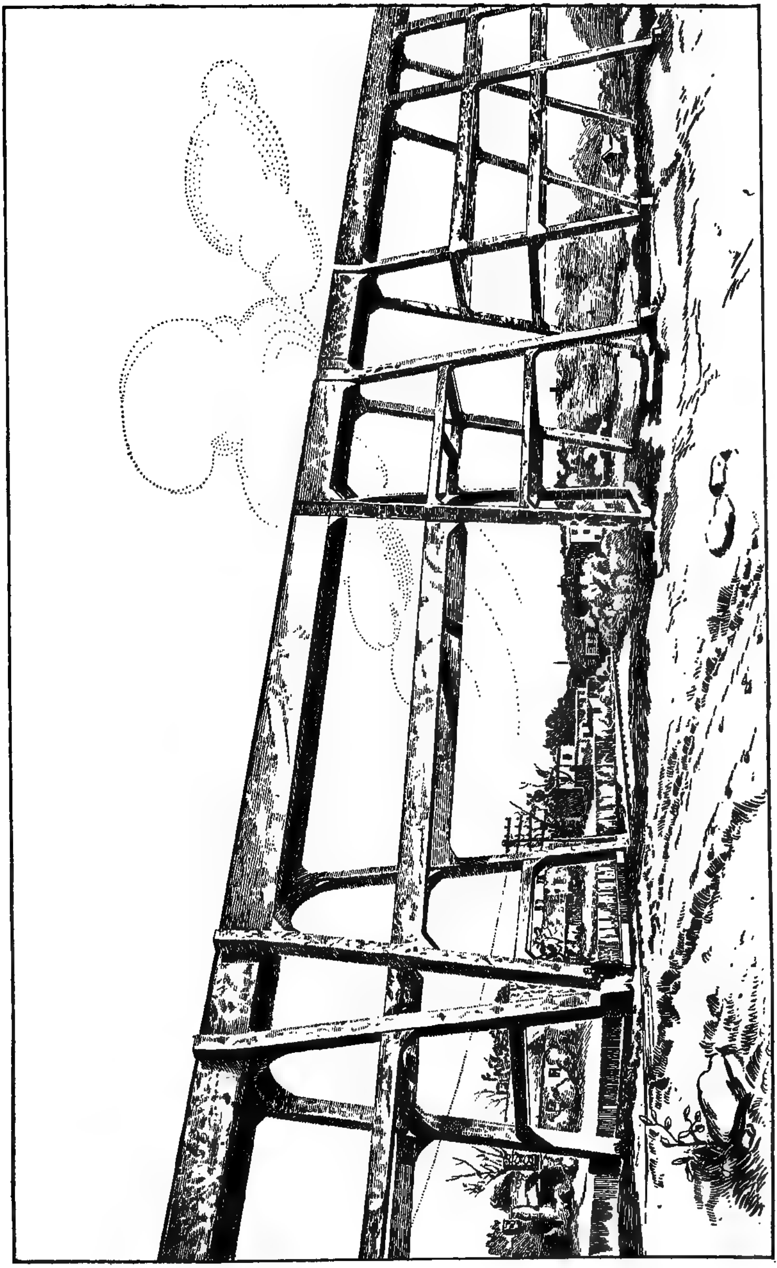


Fig. 28

require painting or other maintenance attention. Reinforced concrete could be conveniently used for the vertical supports and main girders of such a roadbed, and thus avoid the use of structural-steel construction entirely.

34. The viaduct of the Richmond and Chesapeake Bay Railway is an interesting example of reinforced-concrete construction. This viaduct, which is constructed entirely of reinforced concrete, is shown in Fig. 28. The illustration shows the crossing of the Seaboard Air Line Railway and a city street. The viaduct is designed to support a train of cars, each of which is 54 feet long, weighs 150,000 pounds, and is supported on four-wheel trucks, located 33 feet from center to center.

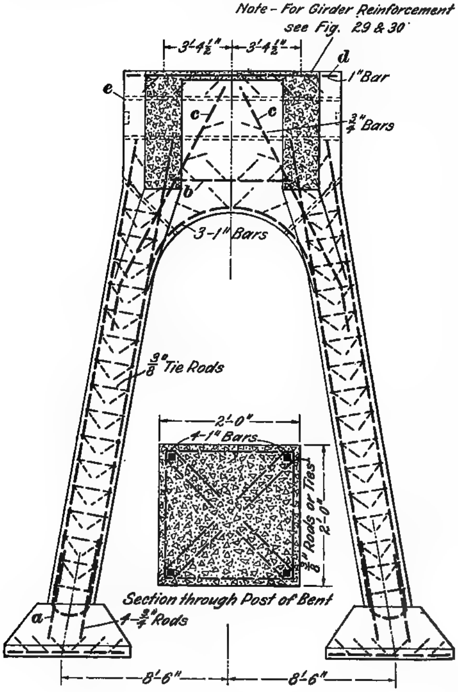


Fig. 29

35. The design of the bents supporting the reinforced-concrete girders that carry this railway over another street crossing is shown in Fig. 29. From this figure, it will be observed that the columns of the bent are reinforced with four 1-inch Kahn bars that are tied with ⅜-inch round iron binders, or ties. In order to make connection to the footings, which were constructed before the upper portion of the bent, short rods or bars *a* were inserted in the footing and allowed to project above the top of the footing. The reinforcing bars in the column bents were afterwards wired or spliced to these rods. The footings are also reinforced in the bottom with Kahn bars, and the bent is strongly reinforced where the columns of the bent are connected at the top by means of a reinforced-concrete portal, or arch.

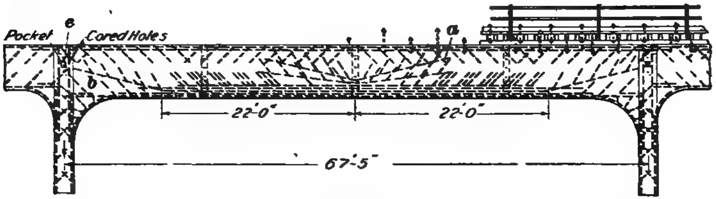


Fig. 30

It will be observed that the portal is reinforced with three 1-inch rods bent around the soffits. To relieve this arch a straight-bar reinforcement is placed over the top, as at *b*, and to reinforce the arch additionally, bracket, or gusset, rods *c* are provided, as are also additional bars *d* near the top of the portal. These bents are connected at the top with heavy reinforced-concrete girders.

The elevation of these girders, shown in Fig. 30, indicates the manner in which the reinforcing bars are disposed. As will be observed, these bars, which are Kahn truss bars, are stopped off in much the same manner as are the plates in the flange of a plate girder. Several of the bars run through and obtain a bearing on the bent, and others are trussed or bent upwards, as shown at *a* and *b*. In order to stiffen the structure, curved gussets, or brackets, are built between the ends of the girders and the bents.

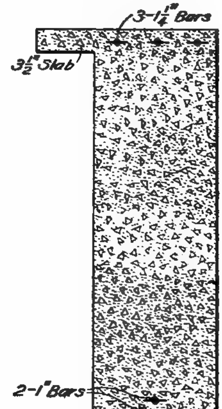


Fig. 31

The span shown in Fig. 30 is the longest one in the construction and carries the roadway across a street. The girders for this span are of the section shown in Fig. 31. They are reinforced heavily at the bottom with two 1-inch Kahn bars and twelve 1¼-inch

Kahn bars, with additional rods in the top consisting of three 1¼-inch Kahn bars. The girders are 70 inches deep and 20 inches wide, and are connected at the top by means of a reinforced-concrete slab. This slab merely forms a shield over the roadway and stiffens the structure laterally at the top.

36. In building the structure, a 1–2–4 mixture of concrete was used, and in making the calculations, a compressive stress of 500 pounds per square inch on the concrete and a tensile stress of 16,000 pounds on the steel reinforcement were assumed. The unit shear was taken at 50 pounds per square inch on the concrete, and 10,000 pounds on the steel. The forms were made of 2-inch lumber, the material being dressed on only one side. Great care was taken to clean the lumber used in the forms as they were taken down, so that smooth work would result when this lumber was used elsewhere in the construction. Generally, the forms were removed on the sides of the girders at the end of the week, but the boards were not removed from the columns nor from the supporting false work beneath the girders until 30 days had elapsed. For the support of the false work of the small girders, 4″ × 4″ studs placed 3 feet from center to center were used. Under the heavier form work, such as at the street crossings, the supports consisted of 6″ × 6″ timbers.

As shown at *e*, Figs. 29 and 30, holes through which ties can be passed or anchor bolts inserted were left in the concrete. This was done to facilitate the work at some future time in case it is desired to double-track the viaduct.

The work of construction was so conducted that traffic was not obstructed on the streets or tracks over which the viaduct passes. It was necessary in several instances to construct truss centering or to brace the centering with oblique struts, so that no struts or false work would come over the tracks or street.

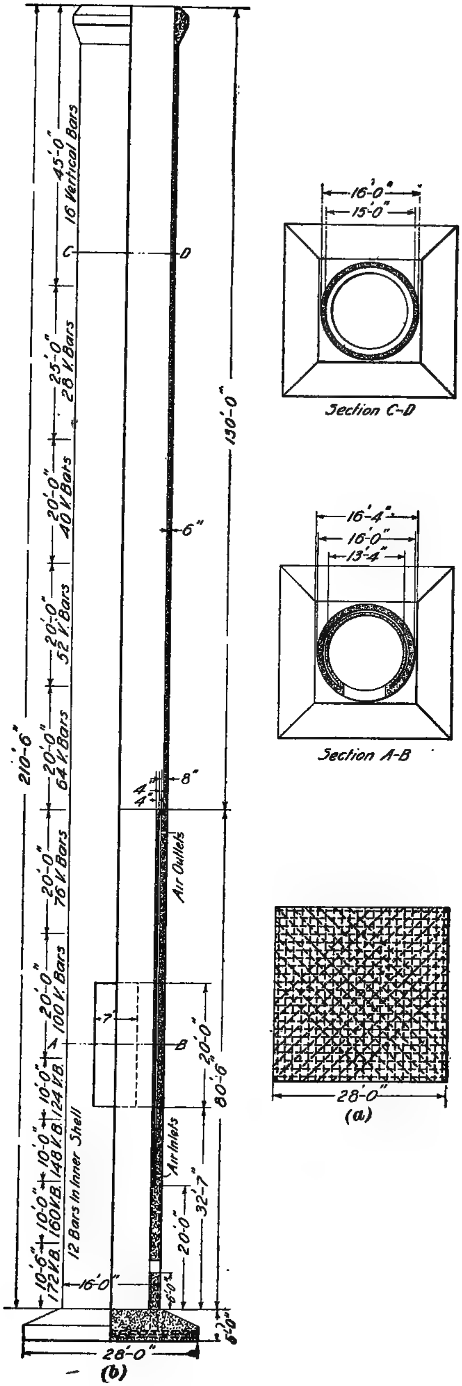

45'-0"
16 Vertical Bars
C
D
25'-0"
28 V. Bars
20'-0"
40 V. Bars
20'-0"
52 V. Bars
20'-0"
64 V. Bars
20'-0"
76 V. Bars
20'-0"
100 V. Bars
10'-0"
124 V.B.
10'-0"
148 V.B.
10'-0"
160 V.B.
10'-6"
172 V.B.
12 Bars In Inner Shell
210'-6"
130'-0"
6"
8"
4"
4"
Air Outlets
7'
A
B
20'-0"
80'-6"
Air Inlets
32'-7"
20'-0"
6'-0"
16'-0"
6'-0"
28'-0"
(b)
16'-0"
15'-0"
Section C-D
16'-4"
16'-0"
13'-4"
Section A-B
28'-0"
(a)

FIG. 32

REINFORCED-CONCRETE STACKS

37. Numerous stacks and chimneys have been constructed of reinforced concrete, and although chimneys built of this material have developed defects, complete failure has occurred in only a few instances. In the design of reinforced-concrete chimneys, a very thin wall, or shell, is constructed, and this is reinforced with vertical rods to resist wind pressure and horizontal rods to resist shrinkage and thermal stresses.

Concrete is not an entirely fireproof material, because it displays a tendency toward surface deterioration if subjected to great heat for a considerable period of time. Consequently, it is generally advisable to build reinforced-concrete chimneys with an inner shelf, or lining, both to lessen the deterioration on the outer shell and to protect the chimney from changes in temperature.

In the construction of reinforced-concrete chimneys, or stacks, it is customary to employ a sectional cylindrical form that is capable of molding a section about 5 feet high. In order to save any additional form work, a very dry mixture of concrete is used that will obtain considerable strength by the next day. In this way, the portion of the work that has been put in place on one day may be used to support the form on the following morning. It is to this method of construction, however, that the principal defects of reinforced-concrete stacks can be attributed, for while they have shown stability to resist wind pressure, they have in many instances shown serious horizontal and vertical cracks after having been erected a short time, many of the cracks following the lines of junction between the different sections of the work. If continuous forms and wet material were used in building reinforced-concrete stacks, their durability would be greatly increased.

38. In Fig. 32 is shown a typical design of a reinforced-concrete stack. According to the dimensions given in view (*b*), the stack is 16 feet in diameter, outside, at the bottom, and 210 feet 6 inches in height from the top of the base to the top of the stack. The base of the stack is 28 feet square and 5 feet thick, and is well reinforced parallel with both sides and diagonally across the corner, as shown in the plan view (*a*).

As shown in view (*b*), the stack is provided with a 4-inch inner shell, there being a 4-inch space between the outer and inner shells. In order that the outer shell may be further protected from the intense heat that is likely to occur directly opposite the flue opening, small air outlets are provided at the top and the bottom of the air space. In building such outlets, care must be taken to see that the inner lining closes fairly tight against the outer shell at the top, because if the cold air enters the outlets and passes into the stack, it will materially affect the draft. The lining used to protect the outer shell is 80 feet 6 inches in height above the base, although it does not start for a distance of 20 feet from the base.

From a study of the section shown in (*b*) it will be observed that the outer shell of the stack to the height of the flue lining is 8 inches thick, while the shell above that point is reduced to 6 inches.

Stacks of concrete may be finished with an astragal, or ornamental top, of any design. Usually, however, the conditions do not warrant anything but a simple band ornamentation as a reinforcement and finish.

From the notes in Fig. 32 it will be seen how the vertical reinforcing bars are arranged. These bars constantly decrease in number from the bottom upwards. The first 10 feet 6 inches in height has 172 vertical bars and the second 10 feet, 160; and, toward the top, the last 45 feet is reinforced with only sixteen vertical bars. The inner shell is reinforced with twelve bars running vertically, and the entire stack is reinforced with horizontal hoop reinforcement, consisting of ⅜-inch bars placed every 6 inches from center to center. Besides the reinforcement mentioned, it is usually well to insert reinforcement around the place where a flue opening enters the stack.

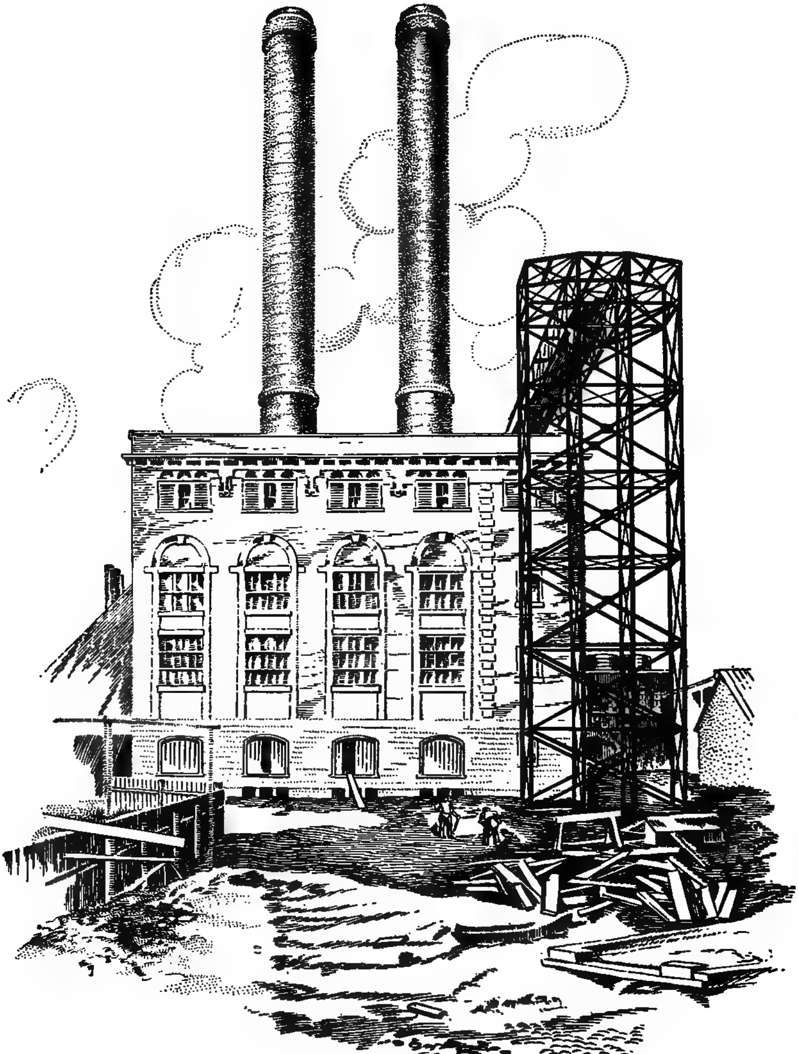


Fig. 33

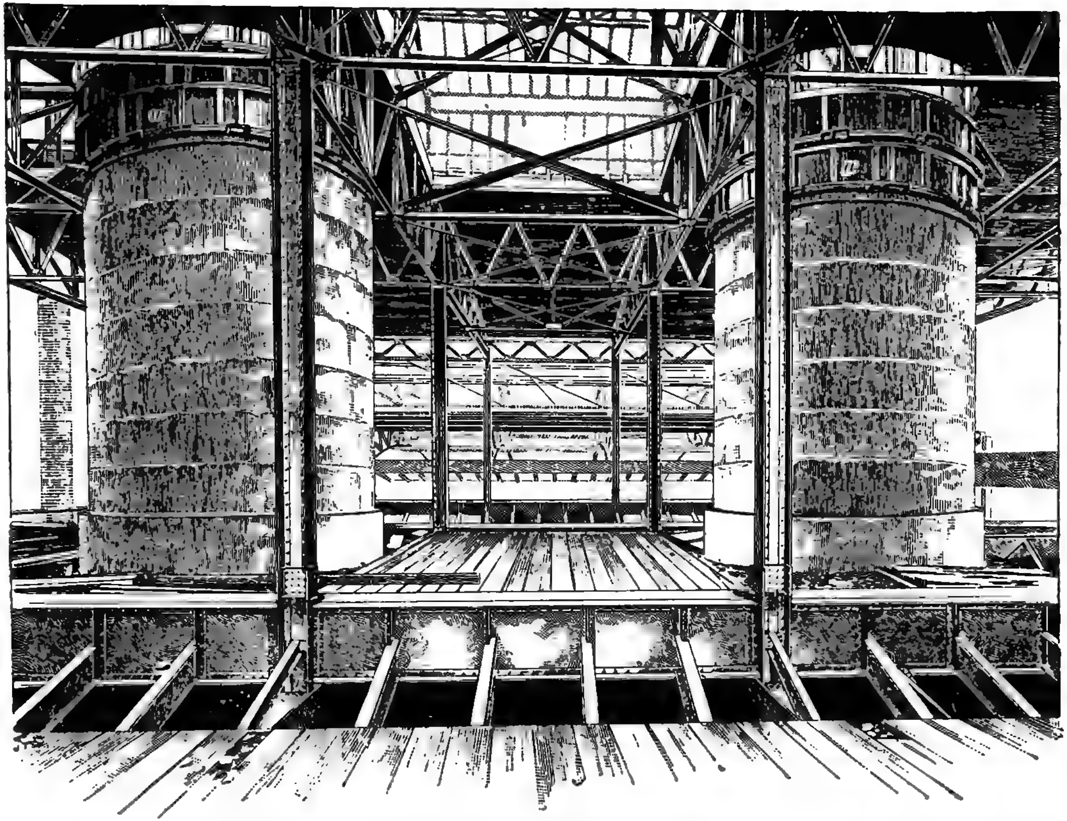


FIG. 34

39. In making the calculations for a stack, its stability against overturning around the base should be determined, as should also its strength to resist bending moment at any section. As the weight of a reinforced-concrete stack is considerably less than a brick stack, it must necessarily have a heavier base, so that the resultant pressure of the wind and weight will fall within the middle third of the base.

In designing the outer shell to resist breaking, as well as overturning from the transverse stress caused by the wind pressure, experience is the best guide. In some instances, the strength of the reinforced concrete in tension is neglected, and only the strength of the reinforced concrete in compression of the leeward side is considered.

40. In many instances, reinforced-concrete stacks are built through a structure, as shown in Figs. 33 and 34, which are views of the power house of the Philadelphia Rapid Transit Company. These illustrations are interesting because they show the character of the work required on a reinforced-concrete stack as well as the method of centering. In Fig. 33, which is an exterior view, two large reinforced-concrete stacks are shown projecting above the roof. Fig. 34 shows the interior of the building, which is of steel construction, and the manner in which the stack passes through the floors and ceiling. At *a* are shown the sectional centerings in position at the top of the reinforced-concrete work, and the several bands, or discolorations, on the stacks indicate the several stages of the work as it has progressed.

REINFORCED-CONCRETE DAMS, AQUEDUCTS, CONDUITS, ETC.

41. Concrete Dams.—Reinforced concrete has been used to some extent for the construction of gravity dams, and in many instances they have been built at a cost much less than that of masonry or timber dams. The general type of these dams consists of reinforced-concrete buttresses, or webs, placed at intervals and supporting a reinforced-concrete slab.

These reinforced-concrete structures, since they lack weight, may, however, be anchored to the rock foundation, where such occurs, with heavy bolts or iron anchors.

42. In Fig. 35 is shown a typical reinforced-concrete dam located at Theresa, New York. The cost of this dam exceeded only slightly the cost of a wooden structure of equal stability. The dam is about 120 feet wide and 12 feet high, and as can be seen from the section, it consists of a reinforced-concrete slab 6 inches in thickness, supported upon reinforced-concrete buttresses 12 inches thick and spaced 6 feet from center to center. The slab is reinforced with expanded metal and with ¾-inch Thacher bars. A very rich mixture of cement was used for the slab portion or spillway, of the dam, while the material used for buttresses and mass work was of a poorer quality. The reinforcement in the slab construction is spaced closer near the foot of the dam, because the pressure at this point is greater, on account of the hydrostatic head. Where the top corner or spillway, of the dam, is formed, it is rounded off so as to make the water flow over in a sheet; in addition, this point is strongly reinforced as a beam.

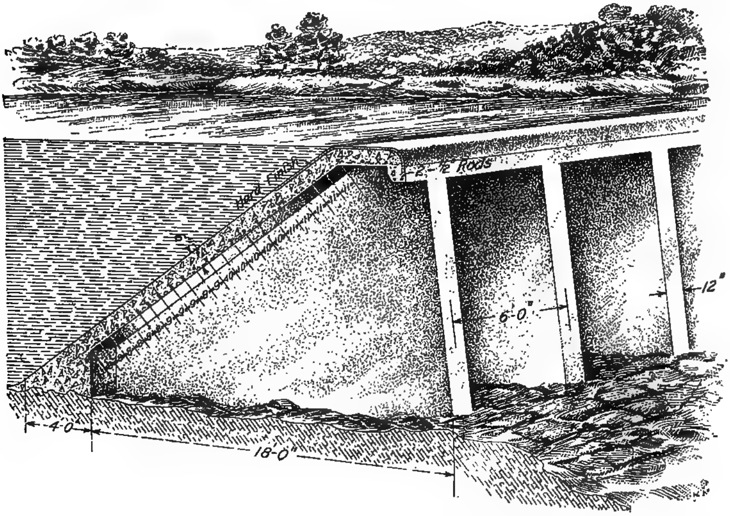


FIG. 35

43. Another method of constructing a dam of this character would be to use a thin slab and support it upon reinforced-concrete beams running between the buttresses.

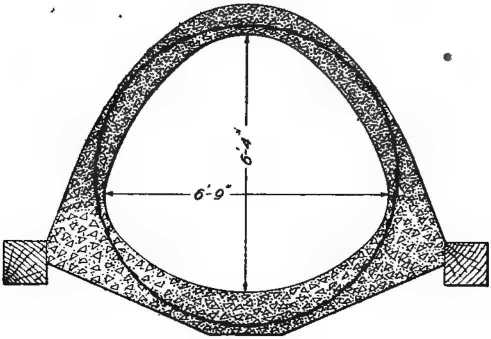


Fig. 36

These concrete beams should be placed closer together near the foot of the dam, and should be pitched farther apart near the upper part, where the pressure is not so great.

The dam shown in Fig. 35 is constructed upon a rock foundation. If the foundation consisted of some other material, a bottom slab, or plate, of concrete, with deep projecting spurs or walls beneath, would have to be provided, so as to prevent the dam from sliding and from being undermined by leakage beneath the structure and by the scouring out of the foundation.

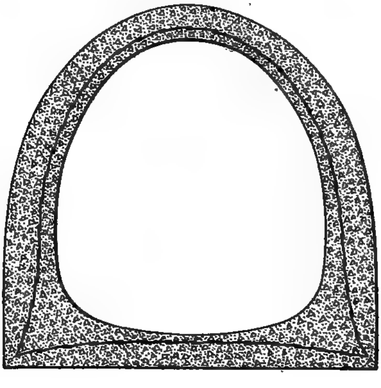


Fig. 37

The construction of dams is a special feature of engineering. It requires the knowledge of experts in this class of construction, because, in addition to being subjected to great hydrostatic pressure, dams are sometimes required to resist the shock caused by floating logs and other material carried down stream, as well as the tremendous force caused by the breaking up of ice jams.

44. Aqueducts, Conduits, Etc.—In addition to structures already discussed, reinforced concrete is used for such constructions as water conduits, sewers, aqueducts, filters, filter beds, and septic tanks for sewage disposal.

45. In Fig. 36 is shown a section through a reinforced-concrete aqueduct constructed for the city of Mexico. This aqueduct is about 17 miles in length, and, as will be observed from the dimensions given on the section, 6 feet 9 inches across at the widest part and 6 feet 4 inches high. The reinforcement of this structure consists of heavy expanded metal, which is in the form of a cylinder and is embedded in the concrete. The section shown is such as will allow a good foundation for the structure. The inside section is arranged so as to reduce the frictional flow of the water, and the top is arched in such a way as to offer great resistance against caving in from the weight imposed on the top of the structure.

Another type of conduit for carrying water is shown in Fig. 37. This conduit was constructed in connection with the Torresdale filter plant at Philadelphia. As will be observed, it is reinforced with expanded metal, and has a form somewhat similar to that shown in Fig. 36.

In nearly all instances, such conduits are constructed with collapsible centering inside and with forms and lagging for the outside work. The work is always carried along in sections.

OFFICE PRACTICE IN CONCRETE DESIGN

DETERMINATION OF DESIGN

FIXED CONDITIONS

1. The method of procedure in the design of reinforced-concrete buildings is as varied as the personality of the architect or the designing engineer. The practice in the offices of different engineers is not in any way uniform, so that while the information given in this Section may not conform to the practice in any particular office, an effort has been made to arrange the calculations and method of design so as to be consistent with the most universally adopted practice, and to systematize these calculations in such a manner as to shorten the work and arrive at satisfactory results.

2. Preliminary Data.—The problem worked out in successive steps in the ensuing articles of this Section will serve to illustrate the current office practice in the design of a reinforced-concrete building. As this type of construction is most suitable for buildings devoted to manufacturing, storage, or commercial purposes, the example will deal with the design of this class of structure.

It is proposed to build a five-story-and-basement factory building 42 feet wide and 146 feet long, outside to outside surface of brick walls, with the entire structural framework of the building of reinforced concrete. The spandrel filling, that is, the space beneath each window sill and above the

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lintel of the window in the story below, is to be of brick masonry 12 inches in thickness. The exterior reinforced-concrete lintels are to be exposed on the face, or, in other words, are not to be veneered, or enclosed, with a brick facing. The piers are to be enclosed in brick, and are to project on the outside, like pilasters, 9 inches beyond the face of the wall.

It is decided that there shall be only one row of columns down the center of the structure. The building is to be used for heavy manufacturing purposes, requiring a floor construction capable of supporting a live load of 150 pounds to the square foot. The greatest amount of glass surface possible is to be provided in the windows, and consideration is to be paid to the construction of the lintels so that the window heads will be as close as possible to the under side of the slab. The building is to be lighted on both the sides and the ends.

Among the special features in the planning of the building is the strengthening of certain columns and wall piers and the roof, in order to form a support for a 25,000-gallon sprinkler tank to be placed on the roof. All the elevator shafts and stairways are to be enclosed with 9-inch brick walls, which are to be supported at each story by the reinforced-concrete construction.

CONDITIONS TO BE DETERMINED

3. Arrangement of Beams With Regard to Light. The general plan of the building is assumed to be as shown in Fig. 1, and the first step in the problem of design is to determine the best layout for the beams and girders; that is, the most economical layout and the one that will work out the best with regard to the lighting facility desired. It is practically determined by the conditions of the problem requiring maximum light that triple windows shall be used between the concrete wall piers, thus making the distance from center to center of wall piers 16 feet.

There are two possible beam framing plans for the building, as indicated in Fig. 2. By running a girder lengthwise of

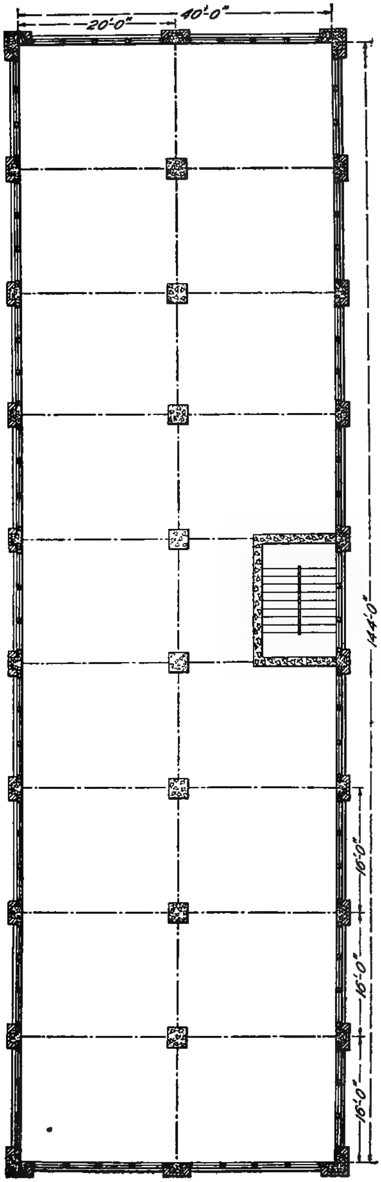

FIG. 1

the building and the beams across the building, lintels of considerable depth are required over the windows in order to receive the ends of the beams, as shown in (*a*). However, if the girders are run across the building, and the beams longitudinally, a lintel of sufficient strength to support only a portion of the floor slab and the spandrel over it will be required at the head of the window, as illustrated in (*b*). It will be noticed that this last system of beam framing will allow the window heads to be placed closer to the ceiling, giving more light in the building, but that the main supporting members or girders of the floor system have the longer span. With the other system, however, the beams, or secondary members, have the longer span and the girders the shorter span.

4. It is a question which of the preceding systems of beam framing is the more economical, and the only way this can be decided for a particular instance is to make the preliminary estimates for the cost of one bay, comparing the two systems at the same unit prices. Before this can be done, however, preliminary calculations must be made to determine the approximate sizes of the beams and girders, as well as the thickness of the floor slab.

It will be assumed that these preliminary calculations are made, and that the sizes of the several beams and girders in both systems of framing have been approximately determined. When estimates of the cost of both systems are compared, it is found that the difference is slight, and as the preference is for the system that will allow the greatest height of window, the system shown in Fig. 2 (*b*) is adopted.

It is desired to follow this same sys-

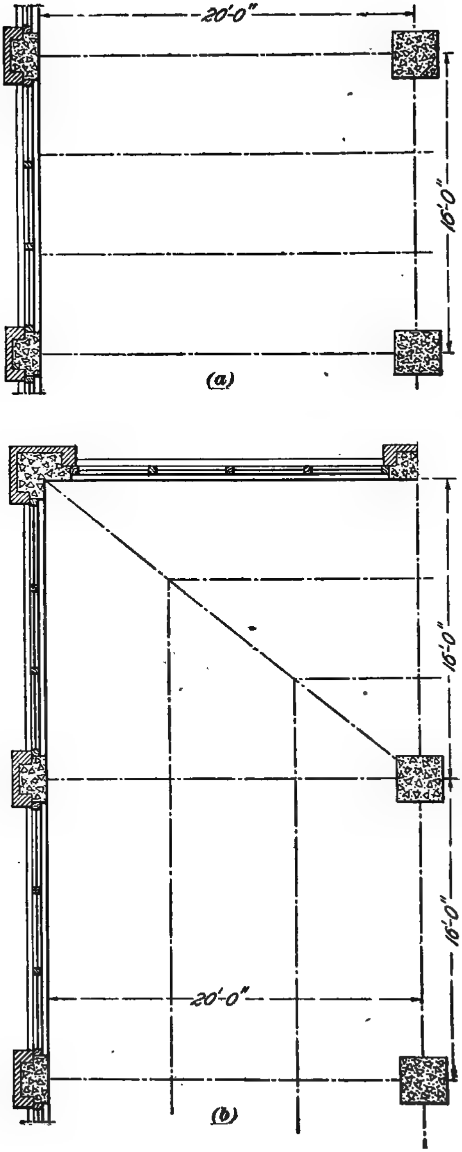

Fig. 2

tem of construction at the ends of the building as well as along the sides, and in order to do so, the end bays at each corner of the building must be formed by using an oblique girder and mitering beams running parallel with the sides and ends of the building into it, as shown in Fig. 2 (*b*). In this manner, the windows at the end of the building will be as high as those on the side.

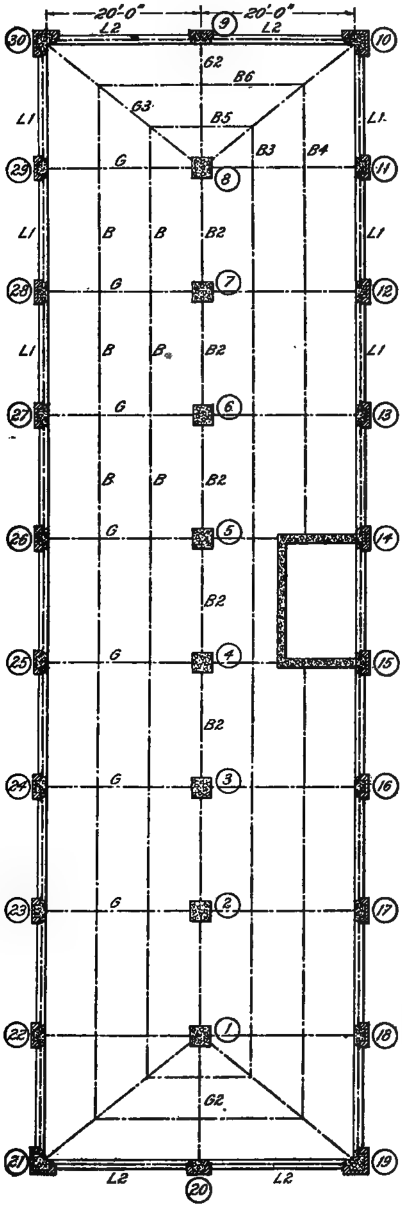

FIG. 3

5. Framing Plan. A typical framing plan of the floor system of the building is shown in Fig. 3. The beams marked *B* are the typical, or normal, beams of the framing plan, while those marked *B* 2 are shorter than the normal beams and those marked *B 3*, *B 4*, *B 5*, and *B 6* are special beams. The normal girders are marked *G*, the girders of the end bays *G 2*, and the special diagonal girders *G 3*. The lintel beams are designated on the framing plan as *L 1* for the normal lintels on the sides, and *L 2* for the lintels on the ends of the building.

The several columns and piers are numbered so that a schedule may be made in which their sizes and reinforcement are given, or by which they may be referred to and identified in marking details. Every office has its own system of marking framing plans that may vary more or less from the above.

CALCULATION OF STRUCTURAL MEMBERS

DETERMINATION OF FLOOR LOADS

6. Live Load.—The live load is generally known from the inception of the structure. It is stipulated by the owners or determined by the architect or the engineer either from data taken by them from buildings used for the same purpose or from information given by the owners. More frequently, however, the live load is fixed by the building laws of the city in which the structure is to be erected.

In the problem under discussion, the live load is taken at 150 pounds per square foot. This load is a good average of the live loads usually employed in designing a factory, although for buildings used for light manufacturing purposes 120 pounds is considered by some engineers as sufficient.

7. Dead Load.—After the live load has been decided on, the next step is to find the dead load. The thickness of the floor slabs and the size of the beams depend on the total load, which includes the dead load. The dead load also depends on the size of the floor slabs and the beams. This state of affairs would seem to prevent the engineer from finding the dead load before the size of the slabs and beams is decided on, and also prevent him from finding the size of the slabs and beams before the dead load is decided on. As pointed out in *Concrete Beam and Column Design*, there are two ways out of this difficulty. The first is by using the tables given for that purpose, and the second is by assuming certain quantities.

8. Some architects and engineers do not use such tables, and in some cases it may be decided to use a certain mixture of concrete for which there is no table arranged. In cases like these, the second method of overcoming the difficulty, which consists in assuming certain values, must be employed. Either the weight of the concrete or the dimensions of the floor slabs and beams may be assumed.

If the weight is assumed, frequently a certain lump figure per square foot, based on precedent, is used. For instance, it is known that the dead weight of a concrete slab and beam system with cinder filling and a wooden floor weighs usually from 100 to 125 pounds per square foot of floor area. The designer would therefore design the slab and beams to carry 100 or 125 pounds per square foot plus the live load. After the correct thickness of slabs and the size of beams have been determined, the designer should calculate their actual weight to determine how nearly correct his first estimate is.

A better alternative is to assume the thickness of slabs and the size of beams and then find their weights. As this is the usual method of procedure for careful designers if tables similar to those mentioned are not to be used, it will be followed here. This method is more accurate than the one in which the weight is assumed first, because with a little experience the dimensions of a floor system will be found easier to approximate than will its weight. Then, again, this method is easier to check, and also the subsequent work can be more or less directed to make the first assumption come true.

9. To show different methods, the clear span has been sometimes used instead of the distance from center to center of supports. This practice was followed by some engineers, but is condemned by the Joint Committee; it is safer and better to follow the Joint Committee.

The distances from the center of the steel to the bottom of the beam or slab are different from those recommended by the Joint Committee. They are put in this Section to represent the practice of different engineers, although the values recommended by the Joint Committee should be used.

10. As an example of the method of calculating the dead load, the floor construction may be assumed to be as shown in Fig. 4. The maple flooring has been decided upon by the architect or the engineer and the owner. It is necessary to use at least 2 inches of cinder concrete to hold the sleepers to which the floor is to be nailed. The thickness of the slabs and the size of beams is accordingly assumed from the experience of the engineer. In the calculations, the weight of the wooden sleepers is neglected.

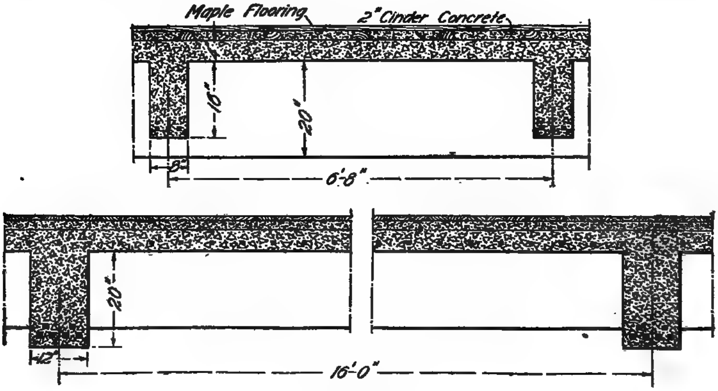

Fig. 4

The dead load per square foot to be withstood by the slab is calculated as follows:

	Pounds per Square Foot
Maple flooring	4
Cinder concrete	18
Concrete floor slab, 4 inches thick	50
Total dead load on slab	72

The beam section is assumed to be 8 inches wide and 16 inches deep, measured from the under side of the slab, so that the section of the concrete beams in parts of a square foot is equal to $.67 \times 1.33 = .891$ square foot. As 1 cubic foot of reinforced concrete weighs 150 pounds, each linear foot of the beam will weigh $150 \times .891 = 134$ pounds. Each beam supports, according to the detail, Fig. 4, a total width of floor of

6 feet 8 inches, measured from center to center of beam, so that the dead load per square foot due to the weight of the beam distributed over this area would equal $134 \div 6.67 = 20$ pounds, approximately.

The dead load in pounds per square foot of floor area on any normal beam is then equal to the following:

	Pounds per Square Foot
Total dead load on slab	72
Dead load due to weight of floorbeam	20
Total dead load on floorbeams	92

From Fig. 4 it is evident that the assumed size of the normal girder is 12 in. × 20 in., so that its weight per linear foot is $1 \times \frac{20}{12} \times 150 = 250$ pounds.

By a calculation similar to that just made for the beam, the added weight due to the girder is almost 16 pounds per square foot, and the total dead load, in pounds per square foot of floor area, is as follows:

	Pounds per Square Foot
Total dead load on slab	72
Dead load due to weight of floorbeam	20
Dead load due to weight of floor girder	16
Total dead load on girders	108

This value, as will be seen later, is used only in the design of columns and not in the design of the girders themselves.

11. Total Floor Loads.—After the live load has been decided on and the dead load has been calculated for each of the elements of the floor construction, the total load for calculating the reinforcement in the slab, beams, and girders may be tabulated as follows:

Floor Slab	Pounds per Square Foot
Live load	150
Dead load	72
Total load	222

FLOORBEAM	POUNDS PER SQUARE FOOT
Live load	150
Dead load	92
Total load	242

FLOOR GIRDER	
Live load	150
Dead load	108
Total load	258

12. Roof Loads.—In reinforced-concrete buildings, it is customary to allow a superimposed load of 30 pounds per square foot for snow and wind pressure combined. With ordinary construction, the dead load per square foot on a roof seldom exceeds from 60 to 70 pounds for the slab, 80 pounds for the slab and beam construction, and 90 pounds for the slab, beam, and girder construction. On these bases, the roof loads for the several structural elements in the roof construction will be as follows:

ROOF SLAB	POUNDS PER SQUARE FOOT
Roof load	30
Dead load	60
Total load	90

ROOF BEAM	
Roof load	30
Dead load	80
Total load	110

ROOF GIRDER	
Roof load	30
Dead load	90
Total load	120

13. Special Loading.—After the several normal floor and roof loads have been determined, the lintel beams should be investigated to find the load that comes upon them, and then any special beams or girders should be analyzed, together with any columns that may differ from the normal column, such columns, for instance, as those which are required to support a tank upon the roof. In designing all such special features of the construction, the greatest care should be exercised in order that in each case the dead load coming upon them may be properly determined, because these special beams and columns are not so likely to have the numerous checks applied to them as are the normal beams, girders, and columns. These special loadings will be discussed when the design of the members that support them are taken up.

DESIGN OF SLABS AND BEAMS

NORMAL MEMBERS

14. Calculations for Floor Slabs.—The total load per square foot upon a reinforced-concrete floor slab connecting beams and girders is not so great as the load per square foot superimposed upon the beams and girders. This was determined by the calculations made in Art. **11.**

The custom in office practice is to make the calculations for the floor slab first, so that if it requires a greater thickness than that assumed in the calculations for the dead loads upon the beams and girders, the assumed load upon these structural members may be adjusted to suit the new thickness of floor slab assumed.

From Fig. 4 it is evident that the distance from center to center of beams is 6 feet 8 inches, and if the beams are 8 inches in width, the clear span of the floor slab is 6 feet. In making the calculations for reinforced-concrete floor slabs, it is customary to consider a portion of the slab 12 inches in width and to figure the load on this width of slab, and likewise to determine the amount of reinforcement necessary in a section of the slab 12 inches wide.

Thus, in the example under consideration, the total load (see Art. **11**) is $1 \times 6 \times 222 = 1{,}332$ pounds, and the bending moment, by the formula $M = \frac{W\,l}{8}$, is $\frac{1{,}332 \times 6}{8} = 999$ foot-pounds, or $999 \times 12 = 11{,}988$ inch-pounds.

As will be observed, in making these calculations, it is more convenient in using the formula for the bending moment to employ the span in feet instead of in inches, but as the bending moment must be determined in inch-pounds, the value found will have to be multiplied by 12.

15. After the bending moment has been found, the slab may be designed by Thacher's, Kahn's, or any other formula. Usually, in office practice, tables are arranged that give the resisting moments of slabs of ordinary thickness. These tables are employed to save the labor incurred by using the formulas every time a slab or beam is to be designed. They are calculated from Thacher's or some other formula. The formula used to calculate the tables depends on the preference of the engineer in charge and also on the recommendations of the building laws of the city in which the building is to be erected. As building laws are often very explicit about the design of reinforced concrete, it is necessary to consult them before going on with the work.

To make the problem under discussion entirely general, the slab will be designed by Kahn's formula. Although the unit stresses used in this formula are fairly high and may not be employed in some cities, yet, on account of its widespread adoption, it will serve the purposes of illustration throughout this Section.

16. Kahn's formula, from *Concrete Beam and Column Design*, is

$$M = 13{,}760\, d\, A$$

in which M = resisting moment, in inch-pounds;
d = effective depth of slab, in inches;
A = area of steel, in square inches.

If a 4-inch slab is to be used, it may be assumed that the depth from the center of the steel reinforcement to the top of the slab, is $3\frac{1}{4}$ inches. In Art. **14**, the required resisting moment was found to be 11,988 inch-pounds. Substituting these values in the equation, it is found that $11{,}988 = 13{,}760 \times 3.25\, A$. Solving for A, it is found that $A = .268$ square inch.

If ⅜-inch square twisted bars are used, the area of each bar will be $\frac{3}{8} \times \frac{3}{8} = .141$ square inch. As the area of reinforcement required for each foot width of slab is .268 square inch, the spacing of the bars will be $12 \div \frac{.268}{.141} = 6.31$, say 6¼, inches.

Kahn's formula stipulates that the reinforcement must be less than 1 per cent. In this case the area is $12 \times 3\frac{1}{4} = 39$ square inches. Then, $39 \times \frac{1}{100} = .39$, which is more than .268. Therefore, less than 1 per cent. of reinforcement is used and the slab is consequently safe. It might be supposed that the reinforcement is so much less than 1 per cent. of the total area that the design would not be economical. Therefore, it might be considered advisable to assume a 3¾-inch slab, calculate the weights over again, and redesign the reinforcement. However, it is the custom of many designers to use, especially in floor slabs, considerably less than 1 per cent. of reinforcement. Although this at first sight would seem uneconomical, yet it has the advantage of reducing the concrete stresses, which is desirable because thin floors are difficult to cast. For this reason, it is decided to use a 4-inch slab reinforced with ⅜-inch square bars spaced 6¼ inches apart.

17. Calculations for Beam Bending Moment.—The bending moment on the beams may now be calculated. From Fig. 2 it will be noted that the span is 16 feet. On account of the width of the girders, for very economical design the clear span might be taken at the actual distance between the girders instead of the center-to-center distance, but this practice, while sometimes used with slab design, as just illustrated, is seldom used with beams of considerable span.

In this case, the total load (see Art. **11**), is $6.67 \times 16 \times 242 = 25{,}826$ pounds, and the bending moment, by the formula $M = \frac{W\,l}{8}$, is $\frac{25{,}826 \times 16}{8} = 51{,}652$ foot-pounds, or $51{,}652 \times 12 = 619{,}824$ inch-pounds.

In making this calculation, as has been mentioned, the distance from center to center of girders was used instead of the actual clear span. As some of the reinforcing rods are bent up over the girders and extend into the beam on the opposite side, the beam may be considered as having fixed ends. In such cases, many engineers find the bending moment by the formula $\frac{W\,l}{10}$ instead of $\frac{W\,l}{8}$. Since, however, the latter is more conservative, it has been used in this problem; that is, the beams will be designed as simple beams.

18. Design of Beams.—Now that the bending moment on the normal beams has been found, these normal members may be designed by any convenient formulas or tables. As Kahn's formula was used to design the slab, it will be used here also. The depth of the beams will be taken to the top of the slab, because the beams and slabs are supposed to be placed at one time and well bonded together by means of stirrups.

As the total depth of the beam is 20 inches, it may be assumed to have an effective depth (from center of steel to top of slab) of 18.25 inches. Substituting the correct values in the formula, it is found that $619{,}824 = 13{,}760 \times 18.25 \times A$; therefore, $A = 2.47$ square inches. The area of the beam is $8 \times 18.25 = 146$ square inches; 1 per cent. of the area of the beam is therefore $146 \times \frac{1}{100} = 1.46$ square inches. The area required is more than this value, but inasmuch as the beam is really of **T** section, the amount of reinforcement will probably not be excessive. The stresses produced should be checked by the method given in *Concrete Beam and Column Design.*

The kind of reinforcement to use is left to the designer. It is of course necessary to use commercial sizes of bars. The combined area of three $\frac{3}{4}$-inch round bars and three $\frac{5}{8}$-inch square bars is 2.497 square inches, which is only slightly in excess of the amount required.

The details of the reinforcement, such as the placement of stirrups, will not be discussed here. These features can be designed by applying the practical rules given in *Concrete Beam and Column Design.*

19. Calculations for Girder Bending Moment. Before attempting to calculate the girder bending moment, it should be borne in mind that the normal girders support their own weight, which, as has been found in Art. **10,** is 250 pounds per linear foot; also, that these girders support the beams at two points, so that each girder may be considered as supporting two concentrated loads.

Referring to Fig. 1, it will be observed that the span from the center of the column to the edge of the wall pier is 20 feet. As the dimensions of the center columns have not been determined, the beams cannot be exactly located on the clear span of the girder. However, it is close enough for practical purposes to consider the conditions to be the same as those diagrammatically shown in Fig. 5.

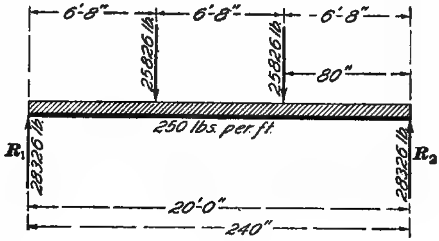

FIG. 5

Since the loads are symmetrical, each reaction is equal to half the total load and the maximum bending moment is in the middle. This bending moment may be found by the ordinary method. Thus, $M = 28{,}326 \times 120 - 250 \times 10 \times 60 - 25{,}826 \times 40 = 2{,}216{,}080$ inch-pounds.

20. Design of Girders.—The total depth of the girder to the top of the slab is 24 inches, and its effective depth may be taken at 22 inches. Substituting the correct values in Kahn's formula, it is found that $2{,}216{,}080 = 13{,}760 \times 22\,A$; therefore, $A = 7.321$ square inches.

This girder is of a **T** section and the above method of calculation assumes that the beam is rectangular. The girder should be finally checked as a **T** beam and for shear, and if necessary the slab should be assisted by steel placed in the top of the girder so as to make it doubly reinforced as well as of **T** section. After the size of columns and piers has been decided upon, the bending moment must be checked and the secondary reinforcement may be placed.

SPECIAL BEAMS

21. Calculations for Lintel Bending Moment. Having figured the slabs and normal beams and girders, the attention of the designer should be directed to the lintel beams along the side walls that carry the spandrel beneath the window sills. The proposed detail of the construction of this portion of the building is shown in Fig. 6, the view in (*a*) being in elevation and that in (*b*) in section. It will be observed from this figure that little benefit is derived from the floor slab acting in conjunction with the concrete of the lintel. The lintel beams, consequently, must be figured not as beams of **T** or **L** section, but as simple rectangular beams.

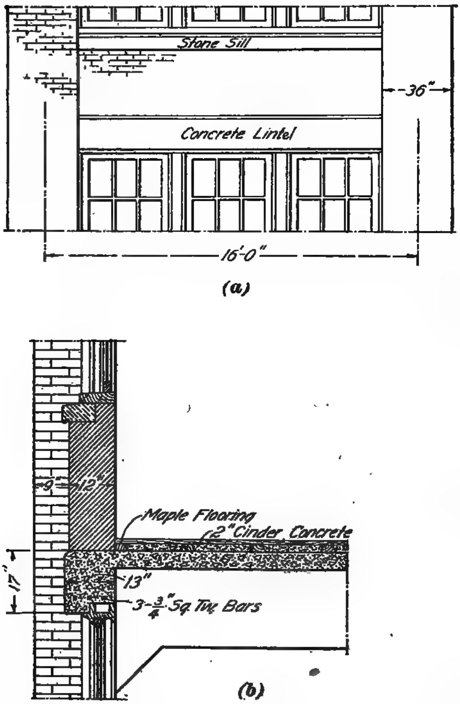

Fig. 6

22. The clear span of the lintel beam, since the piers are 36 inches in width, is equal to the difference between 16 feet and 3 feet, or 13 feet. It will be considered that the lintel beam supports a portion of the slab load equal to one-half the span of the slab extending from the wall to the first beam, and that besides it supports the stone sill and all the brickwork forming the spandrel beneath the window. The calculations will therefore be as follows:

	POUNDS
Floor load $= \frac{6.67}{2} \times 13 \times 222$ (see Art. 9) . . .	= 9,625
Spandrel load, including weight of lintel itself (approximately)	= 6,500
Total load	= 16,125

The maximum bending moment is, therefore, $\frac{16{,}125 \times 13}{8}$ $= 26{,}203$ foot-pounds, or $26{,}203 \times 12 = 314{,}436$ inch-pounds.

23. Design of Lintel Beams.—As rectangular beams are most economical when the steel used for their reinforcement is fully stressed and the concrete is not overstressed, but is stressed up to the allowable stress, an effort should be made in designing ordinary beams to approach this condition. The percentage of steel required for these ends varies with the nature of the concrete, as well as with the ratio of the moduli of elasticity of the concrete and steel. According to Kahn's formula, 1 per cent. of steel is allowable; but, let it be assumed that a somewhat lower stress in the concrete is desired, which would reduce the allowable percentage of steel. In this case, therefore, the desired percentage of steel will be taken at ¾ of 1 per cent.

The width of the lintel is governed by the thickness of the brick wall, and as the object is to make the lintel project slightly from the wall, it will be made 13 inches wide. Its effective depth may be called d. The area of steel required is then approximately $\frac{3}{4} \times \frac{1}{100} \times 13\,d$. Substituting the correct values in Kahn's formula it is found that $314{,}436 = 13{,}760 \times \frac{3}{4} \times \frac{1}{100} \times 13\,d \times d$. Therefore, $d^2 = 234.37$, or $d = 15.31$. It is thus evident that the total depth of the lintel would be about 17 inches.

The area of steel required is then $\frac{3}{4} \times \frac{1}{100} \times 15.5 \times 13 = 1.51$ square inches, which is about the area of three ¾-inch square twisted bars.

24. Design of Miscellaneous Beams and Girders. The main features of the design for the reinforced-concrete

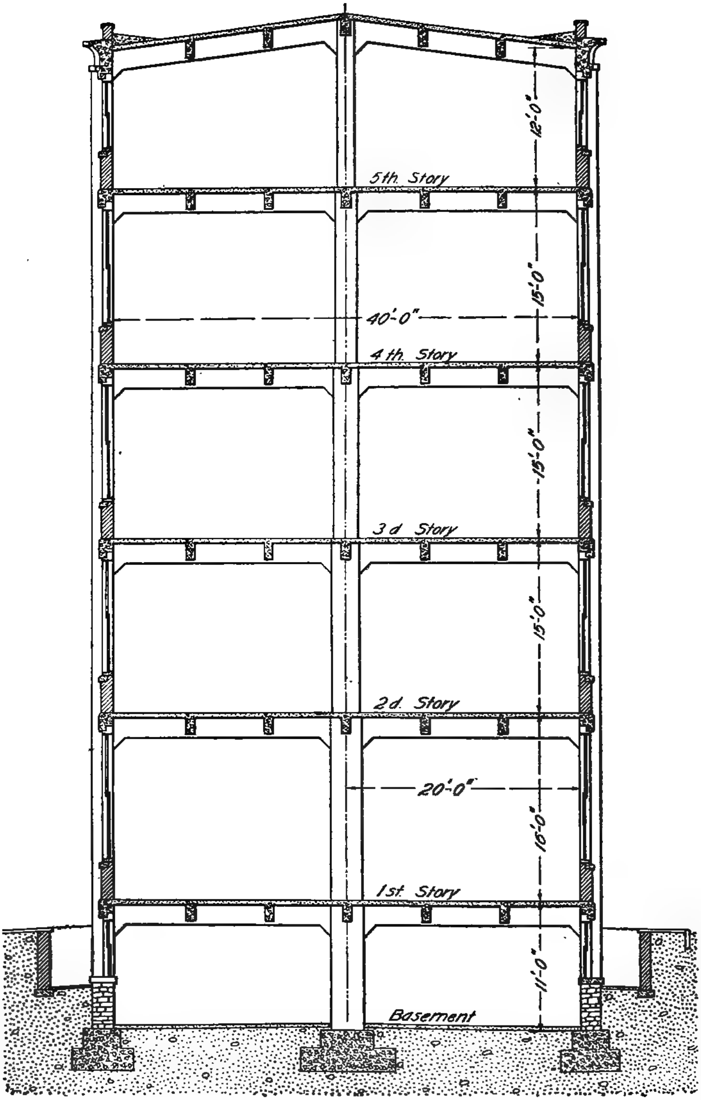
5th. Story
4 th. Story
3 d Story
2 d. Story
1st. Story
Basement
40'-0"
20'-0"
12'-0"
15'-0"
15'-0"
15'-0"
16'-0"
11'-0"

FIG. 7

beams and girders used in the building under discussion have been worked out in a manner consistent with the usual office practice, and it remains only to work out the sizes and steel reinforcement for the roof and other special beams and girders, such as those carrying the water tank, and to decide on such details as the spacing of the stirrups, ties, shrinkage rods, etc., and such secondary reinforcement as may be required to complete the design. These details, with the exception of the stirrup spacing, are usually laid out according to the judgment of the designer. A method of spacing stirrups is considered in *Concrete Beam and Column Design.*

DESIGN OF COLUMNS

NORMAL COLUMNS

25. In calculating the loads upon the columns, it is best to tabulate the figures in such a way that the load on the column of any floor may be determined at once by inspection. The framing plan of the building under consideration is shown in Fig. 3, and the sectional elevation is illustrated in Fig. 7.

From the framing plan, it will be observed that the total area on one floor supported by a normal column is 16 ft. × 20 ft., or 320 square feet, and that the roof area is practically the same.

26. In office practice, it is customary to use a table that gives the strength of reinforced-concrete columns of different sizes. These tables may be calculated by means of any suitable formula. It has been decided in this case to allow a given number of pounds of safe load per square inch of column section, as suggested by the Philadelphia building laws.

Table I gives the safe load on square reinforced-concrete columns for various unit stresses. It will be observed on reference to the table that both the weight for 1 linear foot of column and the amount of concrete required for 1 linear foot of column are given. These values will be found useful in

TABLE I

SAFE LOAD OF REINFORCED-CONCRETE COLUMNS OF SQUARE SECTION, IN THOUSAND POUNDS

Weight for 1 Foot in Height Pounds	Cubic Yard of Concrete in 1 Foot in Height	Size Inches	Area Square Inches	Unit Compressive Stress, in Pounds			
				350	500	650	750
66.7	.016	8×8	64	22.4	32.0	41.6	48.0
84.4	.021	9×9	81	28.4	40.5	52.7	60.8
104.2	.026	10×10	100	35.0	50.0	65.0	75.0
126.0	.031	11×11	121	42.4	60.5	78.7	90.8
150.0	.037	12×12	144	50.4	72.0	93.6	108.0
176.0	.043	13×13	169	59.2	84.5	109.9	126.8
204.2	.050	14×14	196	68.6	98.0	127.4	147.0
234.4	.058	15×15	225	78.8	112.5	146.3	168.8
266.7	.066	16×16	256	89.6	128.0	166.4	192.0
301.0	.074	17×17	289	101.2	144.5	187.9	216.8
337.5	.083	18×18	324	113.4	162.0	210.6	243.0
376.0	.093	19×19	361	126.4	180.5	234.7	270.8
416.7	.103	20×20	400	140.0	200.0	260.0	300.0
459.4	.113	21×21	441	154.4	220.5	286.7	330.8
504.2	.124	22×22	484	169.4	242.0	314.6	363.0
551.0	.136	23×23	529	185.2	264.5	343.9	396.8
600.0	.148	24×24	576	201.6	288.0	374.4	432.0
651.0	.161	25×25	625	218.8	312.5	406.3	468.8
704.2	.174	26×26	676	236.6	338.0	439.4	507.0
759.4	.187	27×27	729	255.2	364.5	473.9	546.8
816.7	.202	28×28	784	274.4	392.0	509.6	588.0
876.0	.216	29×29	841	294.4	420.5	546.7	630.8
937.5	.231	30×30	900	315.0	450.0	585.0	675.0
1,001.0	.247	31×31	961	336.4	480.5	624.6	720.8
1,067.0	.263	32×32	1,024	358.4	512.0	665.6	768.0
1,134.0	.280	33×33	1,089	381.2	544.5	707.9	816.8
1,204.0	.297	34×34	1,156	404.6	578.0	751.4	867.0
1,276.0	.315	35×35	1,225	428.8	612.5	796.3	918.8
1,350.0	.333	36×36	1,296	453.6	648.0	842.4	972.0

making calculations concerning columns, although the weight of columns is often neglected in calculating their strength.

27. The loads upon the columns in the problem under discussion, together with the corresponding sizes required to carry the loads, as obtained from Table I, are arranged as shown in Table II. The allowable unit stress is taken at 500 pounds per square inch.

TABLE II

CALCULATIONS FOR NORMAL COLUMNS

Story Locating Column	Area Carried by One Column Square Feet	Total Load per Square Foot Pounds	Total Load on Column for One Story Pounds	Total Load, Including Load From Column Above Pounds	Size of Column From Table I Inches
Fifth........	320	120	38,400	38,400	9×9
Fourth......	320	258	82,560	120,960	16×16
Third.......	320	258	82,560	203,520	21×21
Second......	320	258	82,560	286,080	24×24
First........	320	258	82,560	368,640	28×28
Basement....	320	258	82,560	451,200	31×31

28. If the preceding method of calculation is to be used, the columns should be reinforced with at least 1 per cent. of steel reinforcement, though, for convenience in handling the steel, nothing less than ¾-inch bars or rods should be used. However, ⅝-inch bars or rods might be used for the 9″×9″ column supporting the roof. Such a small column is very seldom used, however, it being customary, on account of the difficulties attending the pouring of the concrete, not to use any column smaller than 10 inches square.

29. Many owners will not permit columns that are as large as 31 inches and 28 inches to be used in the basement and the first floor, respectively, in which event it will be necessary to use steel cores in these columns. In figuring the steel cores, the concrete is not considered as resisting any compression, and the steel is proportioned by allowing a given safe unit compressive stress. This unit stress is generally taken at 16,000 pounds, so that for the basement columns, the sectional area required for the steel core will be equal to 451,200 ÷ 16,000 = 28.2 square inches, and for the first-story column, 368,640 ÷ 16,000 = 23 square inches. These areas can be made up of any convenient steel sections and then be surrounded with concrete to make a column that is, say, 18 inches square.

SPECIAL COLUMNS

30. Columns Supporting Tank.—Besides the normal columns just discussed, there are many special columns, such as those which support the water tank on the roof and the wall piers themselves.

The columns used to carry the sprinkler tank will be considered first. It is necessary to decide where this tank is to be placed. Let it be assumed that the tank is to be located on the roof and at the corner of the building, and that it is to be 20 feet long and 16 feet wide. Then, three corners of the tank will be supported by wall piers, and the fourth corner will be supported by one of the central columns of the building. The tank is to contain 25,000 gallons of water, which will weigh about 208,000 pounds.

The weight of the tank itself cannot be determined until it is designed. It may be estimated, however, at 150,000 pounds. The total weight of the tank when filled with water is therefore 358,000, say 360,000, pounds. Since each column supports one-fourth of this amount, the interior column will carry 360,000 ÷ 4 = 90,000 pounds. This load is in addition to the load that the normal column carries, as given in Table II. Although the column under consideration is not exactly a normal column, since it does not support normal girders, yet it supports the same floor area, so that its load is practically the same as that of a normal column.

Table III is then formed in practically the same manner as was Table II, giving the total loads at the different stories that the interior column that carries one corner of the sprinkler tank must support.

If desired, the size of this column may be reduced in the lower stories by putting in a steel core.

The columns for the other corners of the tank may be designed in the same way; that is, by taking the loading of the wall pier under normal circumstances and adding to it one-fourth the load of the tank. This operation will not be worked out here, however, as it should present no difficulties.

TABLE III

CALCULATION FOR INTERIOR COLUMN CARRYING SPRINKLER TANK

Story Locating Column	Floor and Roof Load Pounds	Tank Load Pounds	Total Load Pounds	Size of Column From Table I Inches
Fifth	38,400	90,000	128,400	17 × 17
Fourth	120,960	90,000	210,960	21 × 21
Third	203,520	90,000	293,520	25 × 25
Second	286,080	90,000	376,080	28 × 28
First	368,640	90,000	458,640	31 × 31
Basement	451,200	90,000	541,200	33 × 33

31. Wall Piers.—The wall piers on the sides will next be considered. These members carry the loads half way to the center column; that is, each normal wall pier supports an area of 10 ft. × 16 ft., or 160 square feet. This area is equal to half the area supported by the middle columns; therefore, the normal wall piers support half the load. Besides this load, the wall piers carry the load of the spandrel filling—one-half a spandrel on each side. As assumed in Art. **22**, this load will be taken as 6,500 pounds. Table IV may now be constructed for the calculation of the normal wall piers. As these piers will not be square, the required area of the piers as given in the last column of the table is found by dividing the load to be carried by the safe unit stress to be used. The width and thickness of the pier have been decided on, as shown in Fig. 6. The pier is to be veneered with brick on the outside.

32. The rabbet for the window frames is determined usually by the size of the frame. Let it be assumed that the frame has been designed and that the cross-section of the pier is as shown in Fig. 8. The area of the concrete in this pier is 378 square inches. As it is not often considered necessary to reduce the area of the pier in the upper stories of a building, the section shown in Fig. 8, with, say, at least 1 per cent. of steel reinforcement, will be large enough for the fifth, fourth, third, and second stories. The pier, however, will not be sufficiently strong for the basement or first floor, and some additional resistance is necessary.

TABLE IV

CALCULATION FOR NORMAL WALL PIERS

Story Locating Column	Load From Spandrel at Each Floor Pounds	Total Load From Spandrel Pounds	Total Floor Load, Being Half Values Given in Table II Pounds	Total Load at Each Floor Pounds	Area of Pier Required at 500 Pounds per Square Inch Square Inches
Fifth........			19,200	19,200	38.4
Fourth......	6,500	6,500	60,480	66,980	134.0
Third.......	6,500	13,000	101,760	114,760	229.5
Second......	6,500	19,500	143,040	162,540	325.1
First........	6,500	26,000	184,320	210,320	420.6
Basement....	6,500	32,500	225,600	258,100	516.2

In this case, a steel core may be used, as it is not very practicable to enlarge the pier itself. As an example, consider the pier in the basement. The load to be carried is 258,100 pounds. If the allowable unit stress in the core is taken at 16,000 pounds per square inch, the area of core required will be 258,100 ÷ 16,000 = 16.1 square inches. This area of steel may be inserted in the form of angles if desired. A convenient method might be to insert four **Z** bars near the center of the pier. The pier on the first floor is designed in the same manner.

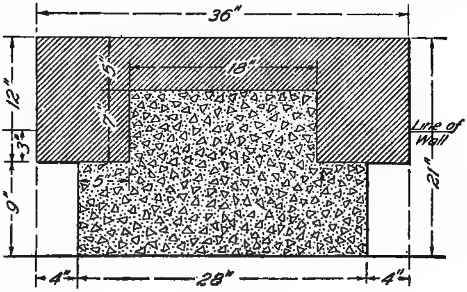

Fig. 8

33. Miscellaneous Columns. After the partition about the elevator and all the stair and elevator details have been laid out, the columns that support these constructive details may be designed by exactly the same methods that have just been used.

The end and corner wall piers are also designed by similar methods.

The cornices and other small constructive details are not usually designed entirely for strength. Many questions arise that experience alone can answer. The designs given in *Reinforced-Concrete Buildings*, Part 1, furnish excellent models for this class of design.

34. The 1½ inches of fireproofing should then be put on the outside of each column. After all the details have been decided upon, the loads on each column, including the weights of the columns themselves, should be recalculated.

REINFORCED-CONCRETE SPREAD FOOTINGS

WALL AND COLUMN FOOTINGS

USE AND ADVANTAGES OF SPREAD FOOTINGS

1. The term **spread footings** is applied to either wall or column footings that have a considerable projection beyond the upper tier of the footing, wall, or column base, as the case may be. Although a grillage foundation composed of a tier, or raft, of steel beams may be rightly termed a spread footing, it is also customary to apply the term to a concrete footing that is comparatively shallow in depth and has such a projection beyond the upper portion of the foundation construction as to require the concrete to be reinforced with steel rods, or bars, in order to resist the bending moment developed.

2. The usual type of spread footing for the support of a column is illustrated in Fig. 1. This footing was designed to be used with a structural-steel column core, and is one that would ordinarily be used in a ten- or twelve-story building in which the usual column spacing is employed, provided the soil were capable of sustaining safely from 4 to 5 tons per square foot.

3. Spread footings are also used in conjunction with the foundations of comparatively low buildings, as those of one, two, and three stories, where the soil is very soft and gravel,

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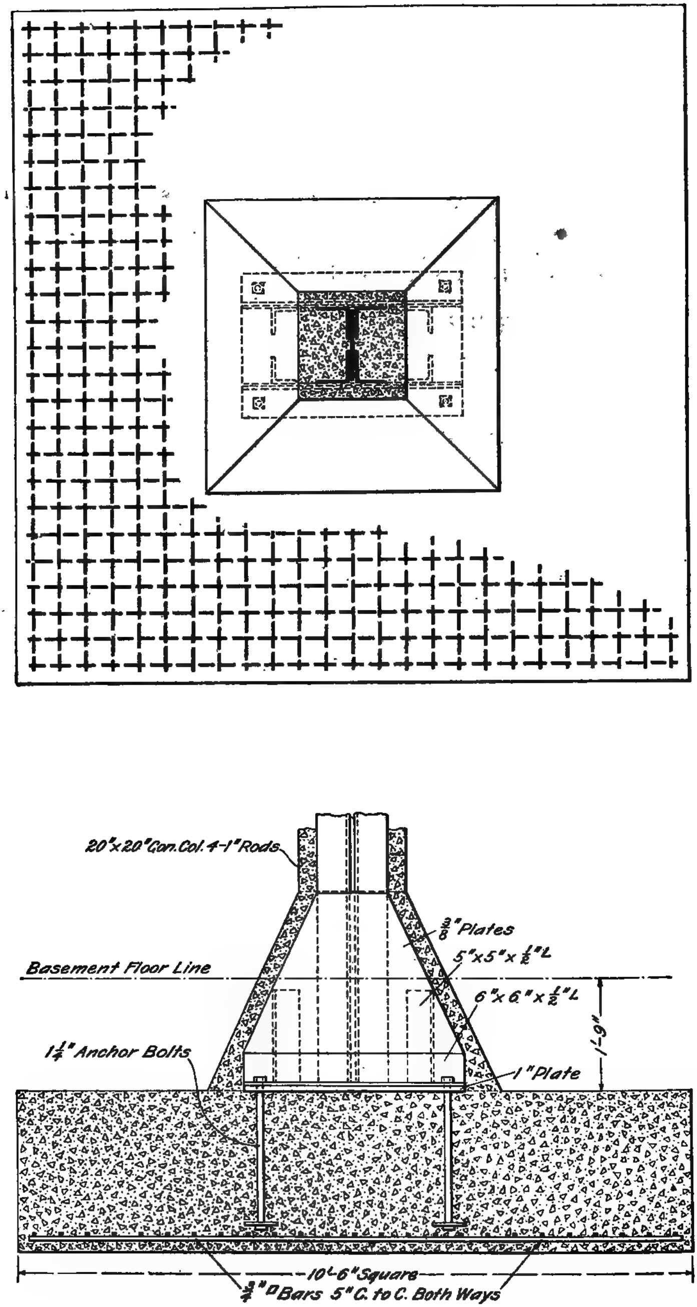

20"×20" Con.Col. 4-1" Rods
⅜" Plates
5"×5"×½" L
Basement Floor Line
6"×6"×½" L
1'-9"
1¼" Anchor Bolts
1" Plate
10'-6" Square
¾" ◻ Bars 5" C. to C. Both Ways

FIG. 1

hard pan, or rock is at considerable depth. For conditions of this character, however, concrete-pile foundations can sometimes be used to advantage.

Spread footings constructed of reinforced concrete are rapidly replacing steel-beam grillages for heavy foundation footings, because they possess distinct advantages over such construction. They can usually be erected somewhat cheaper and they overcome the delay in waiting for structural-steel beams frequently encountered in building construction. In addition, they are not subject to corrosion, which might occur with steel-beam construction under unfavorable conditions or because of imperfect means of protection.

For buildings that are to be constructed entirely of reinforced concrete, spread footings of the same material are particularly desirable, because the reinforcing rods of columns and piers can be made to bond in the concrete of the footings either directly or through projecting vertical splice rods, or bars.

One distinct advantage of spread footings over foundations of the same area as ordinarily constructed is that the vertical height of the footing and pier from the under side to the bottom of the column base is much less; thus excavation is saved, which is an important item of cost where the soil consists of wet clay or some other material of a troublesome nature.

DESIGN AND CONSTRUCTION OF SPREAD FOOTINGS

4. Spread Footings for Walls.—If buildings are to be erected on soil of a very unreliable nature, it is frequently advisable to use continuous reinforced-concrete spread footings under the walls. Such a type of footing construction is shown in Fig. 2, which illustrates, at *(a)*, a section through the footing and, at *(b)*, an elevation of the footing.

Footings of this kind are nearly always employed where the soil is not only soft and yielding, but is also not uniform throughout in its bearing capacity. It is therefore best to design the footing so that its projection will have sufficient steel reinforcement, and that the footing itself will be reinforced with rods extending parallel with the walls of the buildings. If the footing is designed with transverse resistance in a direction parallel with the wall, then, should local soft spots of less bearing value than the rest of the soil occur, the footing will span such places, distribute the load uniformly, and prevent unequal settlement, and, consequently, unsightly wall cracks.

In Fig. 2 (*a*), the reinforcing rods employed to resist the transverse stress in the projection of the footing are shown at *a*; their size and spacing may be determined by applying formulas **1** and **2**, Art. **21**. The rods used to reinforce the footing lengthwise are shown at *b* and *c*. It will be observed that in order to provide a sufficient section of concrete to resist a heavy bending stress, the upper tier of the foundation is carried up in concrete and the bars are placed at the top and bottom of the section. As the lower tier of the footing is of greater width than the upper tier, the lower rods for reinforcement lengthwise of the footing may be spaced at greater distances across the section. They also act as shrinkage rods for the entire lower tier of the footing. These rods in the illustration are $\frac{5}{8}$-inch square twisted bars.

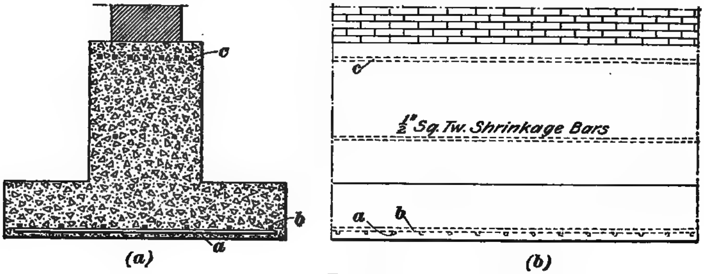


FIG. 2

5. Reinforced-Concrete Column Footings.—Column footings of concrete are sometimes designed with several steps, or layers, of concrete, as shown in Fig. 3, after the manner of a masonry or brick foundation pier. This method of designing such footings is usually undesirable, because considerable concrete is required, and besides it is necessary to excavate to a greater depth than where a typical spread footing of reinforced concrete, as illustrated in Fig. 4, is used. The type of footing shown in Fig. 3 need not, however, be reinforced with steel if the angle made by the line *x x* with the bottom bed of the footing is 60° or more. Where the concrete can be placed at a low cost, this design of footing is sometimes adopted, especially for light loads and ground of a more or less stable nature.

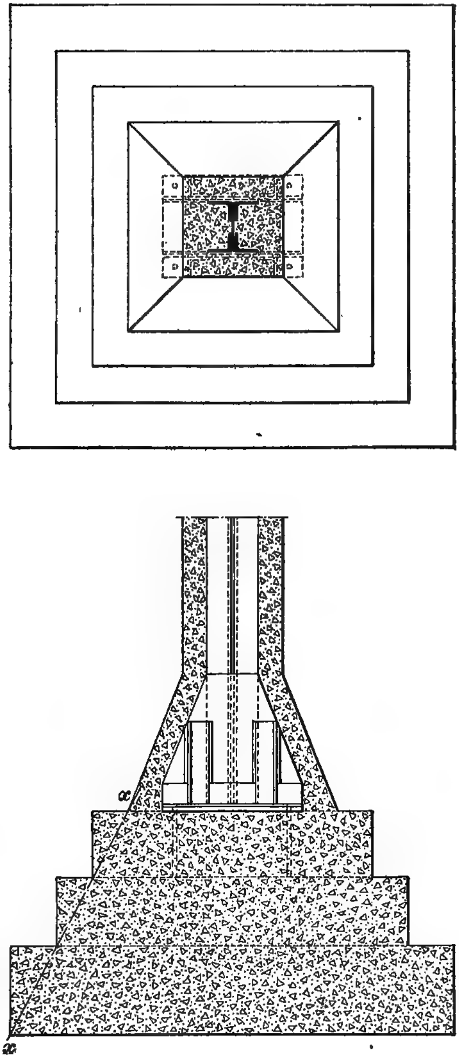


FIG. 3

6. The usual reinforced-concrete column footings, such as shown in Fig. 4, are made from 8 to 20 feet square and are designed to support loads from 200,000 to 1,000,000 pounds. They are used with all conditions of foundation soils, from a stiff clay to gravel and sand or hard pan.

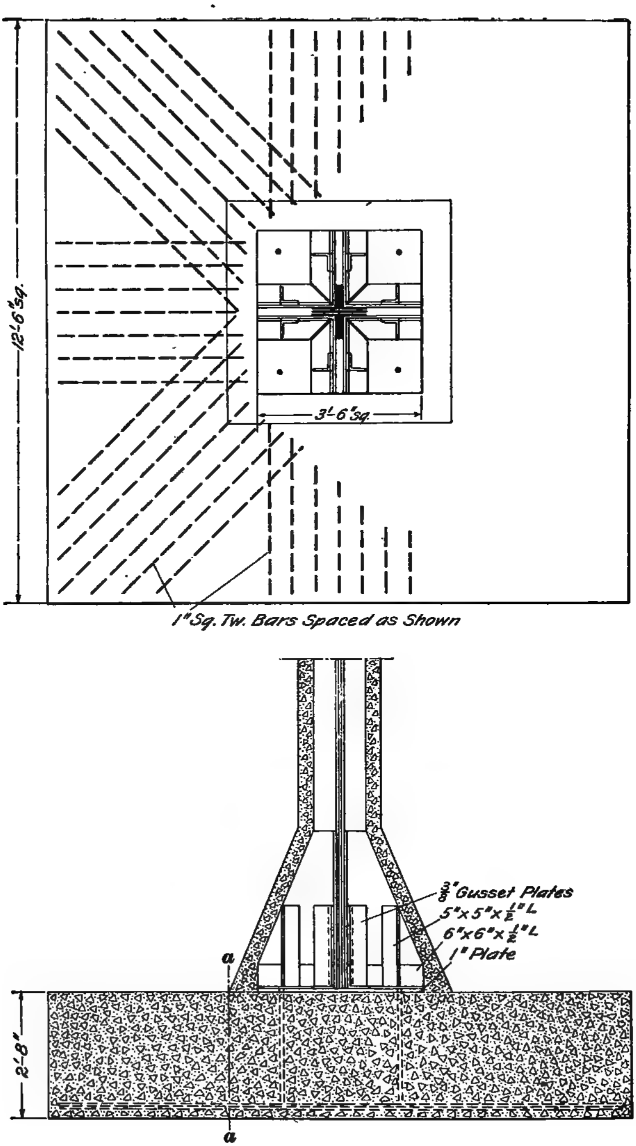

12'-6" Sq.
3'-6" Sq.
1" Sq. Tw. Bars Spaced as Shown
3/8" Gusset Plates
5"x5"x1/2" L
6"x6"x1/2" L
1" Plate
a
a
2'-8"

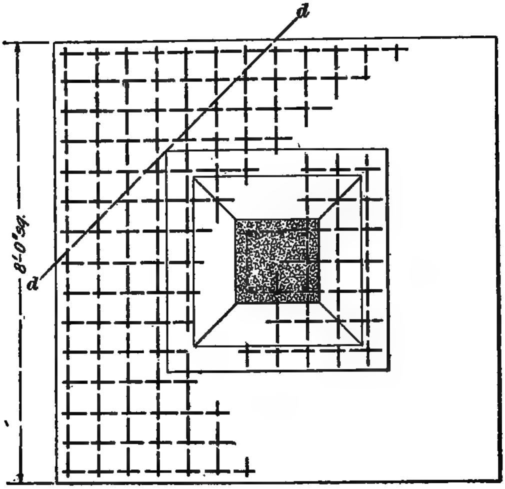
Fig. 4

The theoretical design of a spread reinforced-concrete footing consists in first determining the total load on the column to be supported by the footing, and then finding the required area of the bottom reinforced-concrete footing or layer of concrete by dividing the column load by the assumed allowable bearing value of the soil. The next thing to find is the area of the base of the column where it bears on the upper tier, or layer, of concrete, for the size of this portion of the footing is fixed by the size of the column base. After the areas of the bottom and top tiers, or layers, of concrete have been ascertained, the projection of the bottom tier beyond the upper is known, and the bending-moment stress on the lower tier of concrete can be calculated. Sufficient steel rods may then be introduced, and the footing made of a depth that will resist this bending moment.

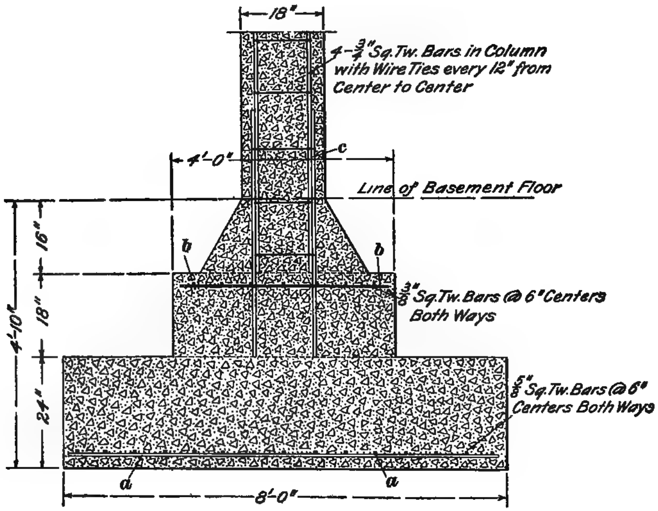
7. In designing reinforced-concrete footings, it is conservative practice to use the same safety factor adopted in the calculations for other parts of the structure. However, there is no danger in assuming tensile stresses in the steel reinforcement of 18,000, or even 20,000, pounds, and unit compressive stresses in the concrete, from bending stress, of 600 pounds, but the direct compression from the column base should not exceed 500 pounds per square inch.

It is not often that the shearing resistance of a reinforced-concrete footing is questioned; in fact, the footing can fail only from this stress through vertical planes in the lower tier, as along the line *a a*, Fig. 4. A failure of this kind is not likely to occur, however, for the following reasons: (1) because of the steel reinforcing rods; (2) because the bottom tier of concrete is usually not less than 18 inches in depth; and (3) because the layer, when calculated and designed for the bending stress, is amply thick to resist the vertical shearing stresses.

As the bottom tier of a reinforced-concrete footing is placed in layers about 6 inches deep, some engineers consider that lines of cleavage in horizontal planes exist, and consequently arrange the reinforcement of the footings with stirrups so as to take care of the horizontal shearing stresses. This practice however, is not general, and usually the horizontal shearing



Plan


18"
4-3/4" Sq.Tw. Bars in Column with Wire Ties every 12" from Center to Center
c
4'-0"
Line of Basement Floor
b
b
3/8" Sq.Tw.Bars @ 6" Centers Both Ways
16"
18"
4'-10"
24"
5/8" Sq.Tw.Bars @ 6" Centers Both Ways
d
a
8'-0"

Elevation

Fig. 5

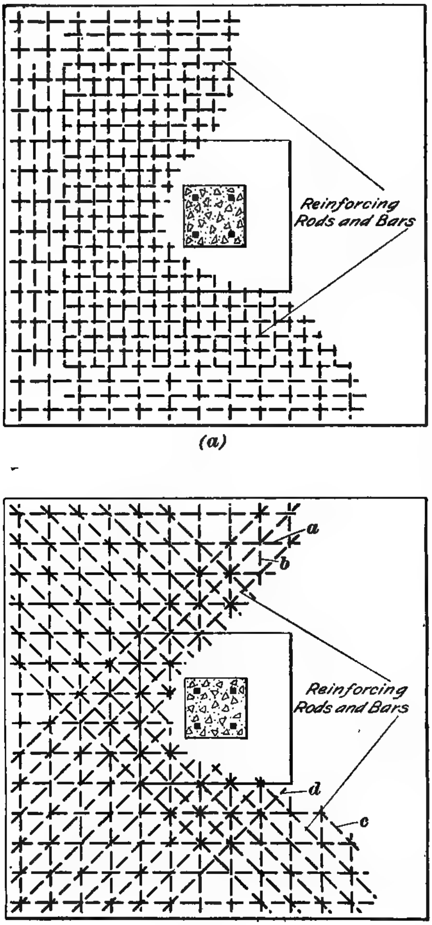
stresses in the concrete are not considered in the design of reinforced-concrete spread footings.

8. Placing Reinforcement in Column Footings. The usual method of placing the steel reinforcement in column footings is illustrated in Fig. 5, which shows the plan and elevation of a typical footing. As the transverse stresses caused by the upward reaction of the soil tend to bend the projection of the lower tier of the concrete footing upwards around the upper tier, or step, of concrete, there is tension created in the lower section of the bottom tier. For this reason, the reinforcing rods, or bars, are placed from 2 to 4 inches from the bottom of the footing, as shown at *a*. The transverse stress occurs on all sides of the footing, and the bars are therefore arranged so as to cross each other at right angles, as shown in the plan. No attempt is made to interlace the bars, or rods; one set is simply laid over the other. It is good practice, however, to wire them together before placing the upper layers of concrete, for by so doing, any danger of misplacing the bars is avoided. Several tiers of bars, or rods, are used in heavy footings, although spread footings that support light loads are sometimes reinforced with expanded metal or woven-wire fabric.

9. In designing column footings in reinforced concrete, it is always good practice to place some steel reinforcement in the upper part of the footing directly under the bearing plate, or base, of the column, as shown at *b*, Fig. 5. This reinforcement is not subject to any bending stress, but acts as a mattress to distribute the concentrated load from the column, and also as a bond to tie the concrete together and consequently increase its bearing resistance.

The footing shown in Fig. 5 is arranged to receive a reinforced column, and for this reason vertical rods *c* are embedded in the concrete. These rods project above the upper surface of the footing for a distance of 3 feet, and to them are wired the column rods. Such vertical rods tend to make a secure and strong junction between the concrete footing and the column, and act in the capacity of anchor bolts, which would be used instead to secure the steel column cores or cast-iron bases.

10. Two other methods of arranging the reinforcing rod in concrete column footings are shown in Fig. 6. In (*a*), the rods and bars are crossed at right angles; every other bar is made short so as to save metal. This method can be used without in any way affecting the strength of the footings, because the bending moment decreases to zero at the outer edge of the footing. If the tranverse resistance at the point where the steel bars are stopped off is thought to be insufficient, the bending moment should be calculated at this point, and, if necessary, the bars extended.



(*a*)

(*b*)

Fig. 6

In the methods of placing the steel reinforcement shown in Figs. 5 and 6 (*a*), it will be observed that the corners of the footings have great leverage about the upper tier of concrete or the column base; also, that the bars, when arranged as shown, are not in a position to resist efficiently any bending stress about the line *d d*, Fig. 5. Therefore, in order to strengthen the corners of the footing, the reinforcing rods are frequently arranged as shown in Fig. 6 (*b*). In this case, the rods cross each other at right angles and extend parallel with the sides of the footings, as shown at *a* and *b*; also, the diagonal reinforcement is arranged in both directions across the corners, as at *c* and *d*, the width of the diagonal tiers being usually made equal to the diagonal dimension of the upper tier of concrete or the column base.

11. The steel reinforcement of column footings is never painted, even if the footings are to be placed in damp situations, because coating the bars will partly destroy the bond that it is necessary to maintain between the concrete and steel; besides, the concrete is sufficient protection for the steel against any serious corrosion. In all instances, however, the steel bars should be cut off enough to allow the ends of the bars to be entirely protected. Thus, as no metal is needed at a distance of from 4 to 6 inches from the edge of the footing, a considerable saving in the weight of the metal can be made.

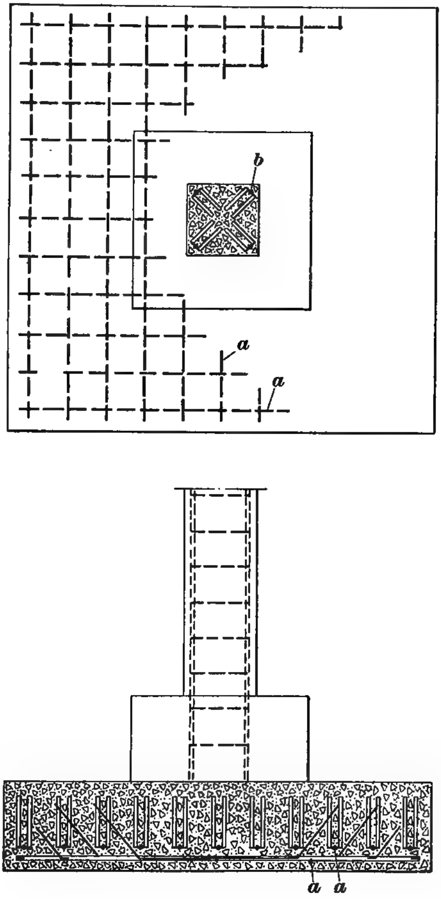


Fig. 7

Plain steel bars and rods are frequently used to reinforce spread footings, and they are probably efficient for all practical purposes. The more conservative practice, however, is to use some type of deformed bar, such as the square twisted, corrugated, or lug bar. In designing a number of column spread footings in a single structure, it is best to space the rods or bars in such a way that the same size can be used throughout. This will simplify the ordering and placing of the bars in the field.

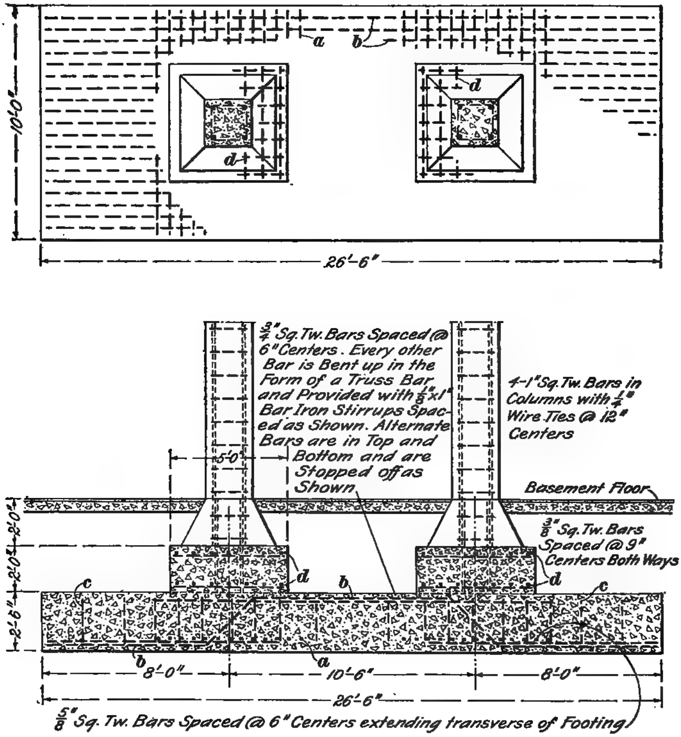
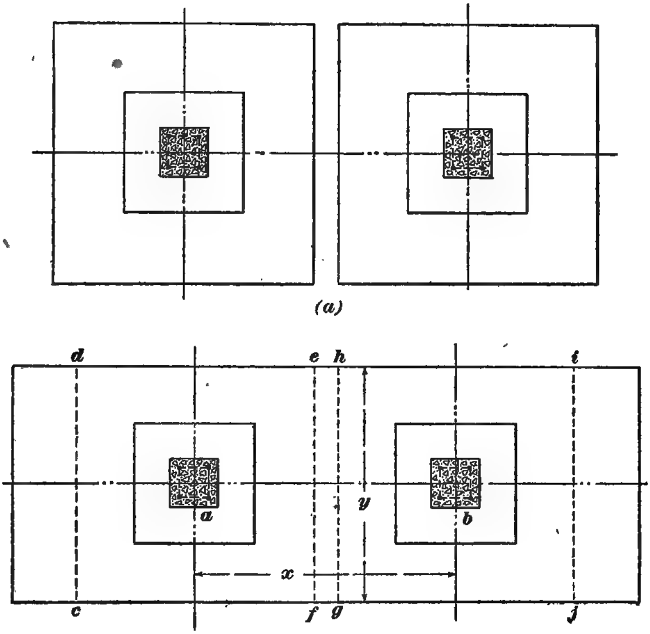

a
b
d
d
10'-0"
26'-6"
3/4" Sq. Tw. Bars Spaced @ 6" Centers. Every other Bar is Bent up in the Form of a Truss Bar and Provided with 1/8"x1" Bar Iron Stirrups Spaced as Shown. Alternate Bars are in Top and Bottom and are Stopped off as Shown
4-1" Sq. Tw. Bars in Columns with 1/4" Wire Ties @ 12" Centers
5'-0"
Basement Floor
3/8" Sq. Tw. Bars Spaced @ 9" Centers Both Ways
2'-0"
2'-0"
2'-6"
c
d
b
d
c
b
a
8'-0"
10'-6"
8'-0"
26'-6"
5/8" Sq. Tw. Bars Spaced @ 6" Centers extending transverse of Footing

FIG. 8

12. Column Footings With Kahn Bars.—The Kahn, or trussed, bar described in a preceding Section is particularly adapted to reinforcing spread footings of concrete, especially if the construction is heavy and if stirrups are considered an advantage.

A column footing reinforced with Kahn bars is shown in Fig. 7. The bars are arranged as indicated at *a*. The stirrup projecting upwards in the bed of the concrete footing tends to prevent any oblique tension or shear cracks, and also acts as a deformation on the bars, giving greater bond in the concrete. The column rods, also of Kahn bars, are indicated at *b* in the plan. These rods, as shown, bond with the footing concrete.



(a)

(b)

FIG. 9

13. Connected Footings for Columns.—The conditions of construction sometimes warrant the combining of the footings under two columns, as shown in Fig. 8. This practice is good only where the area available for the footings is so limited as to prevent the use of separate spread footings. The idea of the combination footing is best explained by means of Fig. 9. In view (*a*) of this figure are shown two columns with separate and distinct footings—the best possible arrangement where the necessary room is available. However, for sake of illustration, assume that the distances x and y, view (*b*), are limited on account of the plan of the basement or the proximity of other columns or features of construction and equipment. Then the footing plan shown in (*b*) is the only one available, for, by reinforcing the concrete between the columns *a* and *b*, the loads are transmitted to the soil uniformly and the area of the footing is increased over the aggregate area of the two square footings indicated by *c d e f* and *g h i j*.

14. The general design of a combination footing, as well as the method of reinforcing it, is illustrated in Fig. 8. It is necessary to reinforce such a footing both across its width and in a lengthwise direction. The transverse rods, or cross-rods, are shown at *a*. These rods may be proportioned and arranged the same as the rods in any footing, and are placed in the bottom, as the projecting footing is subjected to tension at the lower portion of the section. The rods that reinforce the footing lengthwise, as at *b*, must be bent upwards as they approach the space between the two columns. This is necessary because the concrete at this point is subjected to tension in the upper side, due to the negative moment caused by the reaction from the soil acting upwards. For these reinforcing rods, stirrups should be provided, as at *c*, and a mattress of steel reinforcement should be embedded in the concrete beneath the bearing of the column, as at *d*.

15. **Spread Foundations of the Raft, or Continuous, Type.**—In many instances where heavy buildings are constructed upon soft soils, reinforced-concrete foundations have been extended continuously under the entire building. Sometimes a solid raft of concrete from 3 to 5 feet in thickness is used, and in other instances attempts have been made to economize in material by using a beam-and-slab design. Where the beam-and-slab construction is used, the beams may be formed on either the bottom or the top. If they are formed on the bottom, the earth, where possible, is excavated so as to act as the forms for beam construction. If the beams, or ribs, are molded on the top of slabs, then the space between them is filled with earth or cinders, on which the basement or ground floor is laid. In all instances it is necessary to reinforce the construction heavily on both the top and bottom, as flexure is likely to occur in both an upward and a downward direction.

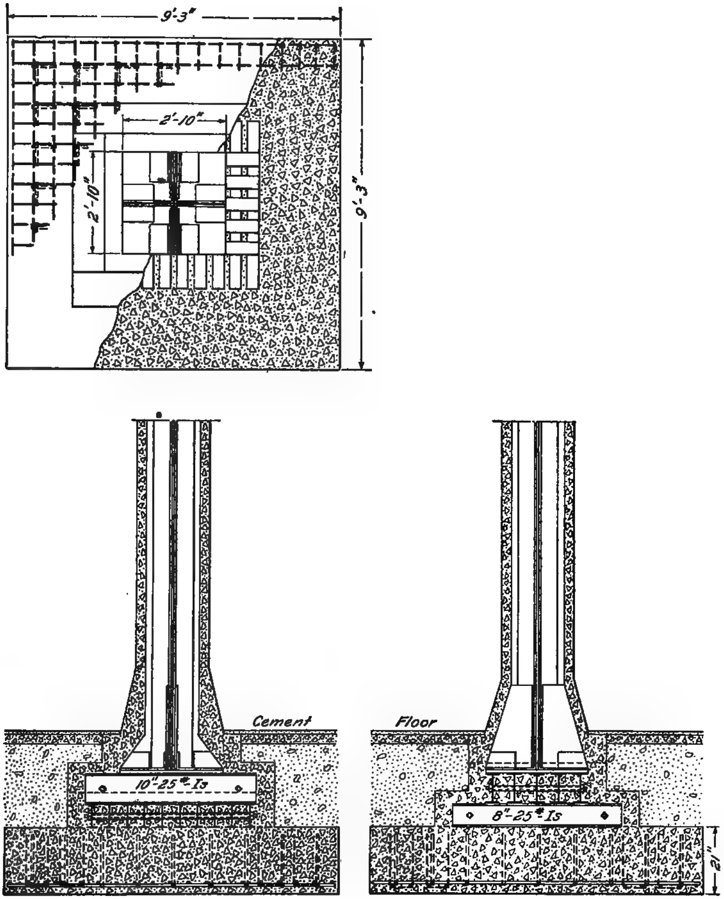


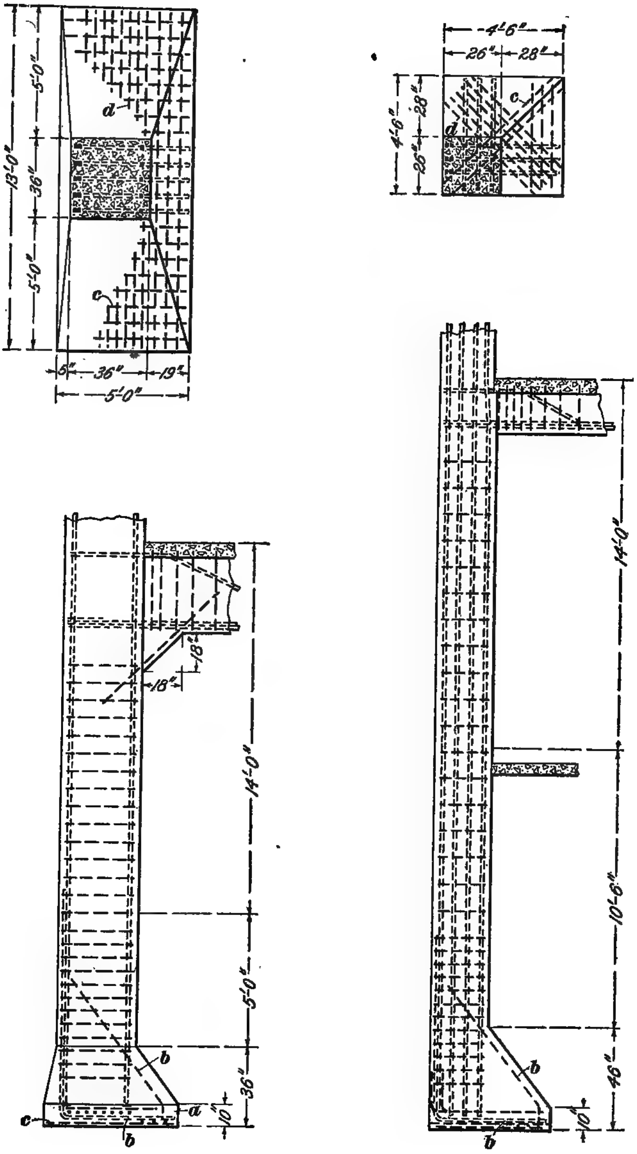
Fig. 10

The type of foundation construction just described has been more extensively used in Europe than in America. It is neither so economical nor so efficient as an isolated footing carefully proportioned for the loads that it carries. There are seldom any conditions that warrant the use of this kind of construction.

16. Spread Footings Combined With Grillage Construction.—If the reinforced-concrete footing is to be very large, and a great projection beyond the column base is not desirable, it is sometimes necessary to combine a grillage-beam construction with the reinforced-concrete footing. In Fig. 10 is shown a typical construction, in which a steel-beam grillage is provided directly under the base of a steel-column core, the grillage resting upon a reinforced-concrete footing. By the use of this construction the base of the steel-column core need not be so large in area; consequently, the footing need not be of such heavy construction, because the steel-beam grillage transmits the load directly to the concrete.

The grillage beams must be of such an area as not to exceed the unit bearing stress upon the concrete reinforced with steel rods, this unit stress being from 500 to 600 pounds per square inch. The steel-beam grillage is designed for bending moment, and is tied together at the ends and intermediately with either standard or pipe separators. If they are to be embedded in the concrete work, the grillage beams are not usually painted.

17. Spread Footings for Outside Columns.—Instead of a continuous footing, isolated spread footings are frequently used in outside, or wall, columns. Such footings are illustrated in Fig. 11. In (*a*) is shown a spread footing for a wall column. Here, the footing is confined on the one side by the party line, and is arranged to project inside the building on the other side of the column, as at *a*. The projecting end of the footing is heavily reinforced as an inverted cantilever, as shown at *b*. Besides this reinforcement, which also takes up considerable shear, rods are run lengthwise of the projection, as shown at *c*, and other rods are extended beneath the column and length-


13'-0"
5'-0"
36"
5'-0"
d
c
5"
36"
19"
5'-0"
4'-6"
26"
28"
4'-6"
28"
26"
c
d
18"
18"
14'-0"
5'-0"
36"
10"
a
b
c
b
14'-0"
10'-6"
46"
10"
b
b

(a) (b)

Fig. 11

wise of the footing, as at *d*. The column rods are well interlaced and wired to the rods of the footings, and the spread end of the footing is so well reinforced and incorporated with the column that the construction will act as a unit in transmitting the load to the soil.

In Fig. 11 (*b*) is shown a spread footing designed for a corner column. This footing is reinforced in a manner similar to the wall column so far as the rods *b* are concerned, but rods crossing in several directions, as shown at *c* and *d*, are also provided to add strength. The position that the footings occupy on the building plan is shown in Fig. 12, the wall footing being at *a* and the corner footing at *b*.

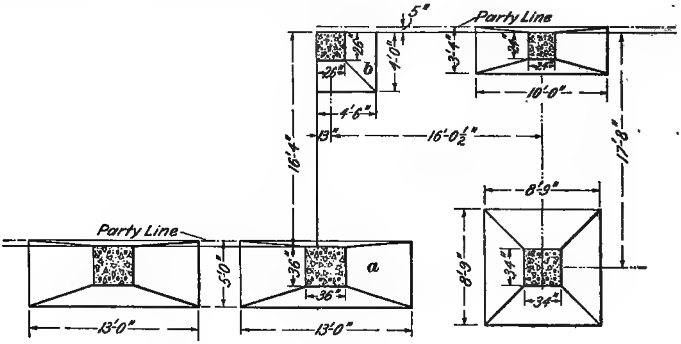


FIG. 12

18. Ribbed Reinforced Column Footings.—Attempts have been made to economize in the construction of reinforced column footings by making the bottom of the footing a thin slab and strengthening it with ribs, or webs, of reinforced concrete. Such a footing is illustrated in Fig. 13, the thin slab of reinforced concrete for a bearing surface on the soil being shown at *a*. A pedestal, or pier, of concrete is incorporated with this slab, as at *b*, from which ribs of reinforced concrete radiate, as shown at *c*. The bottom slab must be reinforced in all directions, so as to transfer the reaction from the soil to the ribs, which are the main supporting members; also, as these ribs are in the nature of inverted cantilevers, they must be well reinforced, with both top and bottom reinforcement, as shown at *d*. This reinforcement must be well tied together with stirrups, as at *e*. It will be observed that the ribs are not particularly efficient as structural members, as the compression from the transverse stress at the top of the rectangular section where the area is reduced necessitates heavy reinforcement. The reinforcement for the slab portion of the footing may be either light rods or woven-wire fabric, and there is a saving here in the weight of the steel required over the reinforcement necessary for other types of footings. This saving, however, is usually more than made up by the steel reinforcement required for the ribs. There is also some saving in the concrete used in this footing, as compared with the solid-slab footings, but this is offset by the labor and material necessary for the form construction. It is doubtful whether a footing designed with ribs, as described, possesses any advantage over other types.

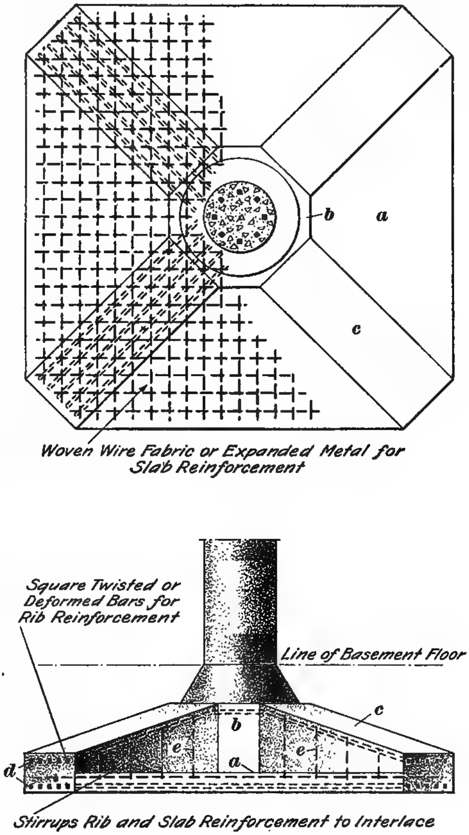

b
a
c
Woven Wire Fabric or Expanded Metal for Slab Reinforcement
Square Twisted or Deformed Bars for Rib Reinforcement
Line of Basement Floor
c
b
e
e
a
d
Stirrups Rib and Slab Reinforcement to Interlace

FIG. 13

19. Column Footings at Different Depths.—In building construction, column footings adjacent to each other and at different depths frequently occur. In such instances, the design of the spread footings should be carefully considered, so that the bearing of the higher footing will not be undermined by putting in the lower one. If a line drawn at the angle of repose of the soil from the higher footing, as at *x x*, Fig. 14, does not intersect the base of the lower and adjacent footing, no precaution need be taken. If, however, the lower footing is so close that the line *x x* does intersect the base, as in Fig. 14, then a footing of special design, as shown at Fig. 15 (*a*), must be resorted to in order to hold the earth back; or, if this kind of footing is not used, the upper footing must be carried down to nearly the same level as the lower one, as shown in (*b*). Such conditions as these are likely to arise in the planning of the footings where basements are to be used as boiler or engine rooms because greater headroom is required in such places and near or around elevator shafts.

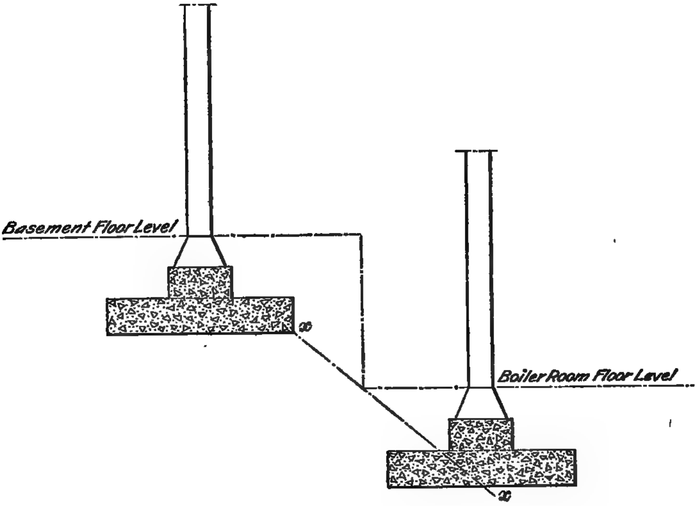


FIG. 14

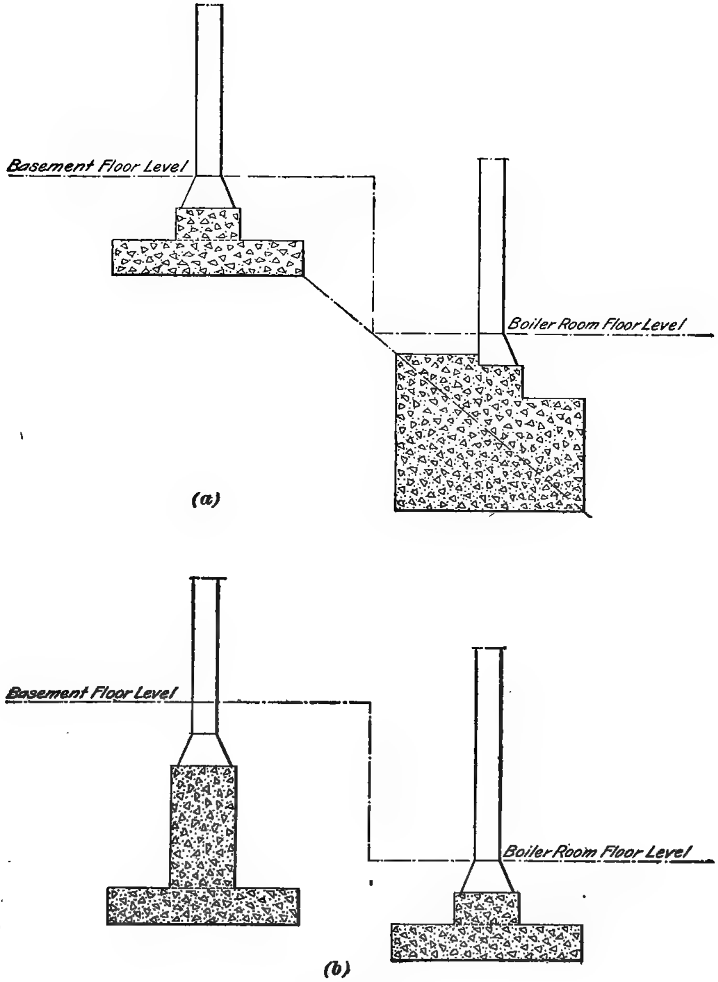

Basement Floor Level
Boiler Room Floor Level
(a)
Basement Floor Level
Boiler Room Floor Level
(b)

FIG. 15

FORMULAS FOR THE DESIGN OF FOOTINGS

20. The usual formulas for bending moments and the resistance of reinforced-concrete rectangular sections may be applied in determining the strength of reinforced-concrete spread footings, and the moments to be resisted may be determined by the methods given in *Heavy Foundations*. However, in office practice, it is desirable to use direct formulas for finding the area of steel reinforcement required and for determining the unit compression created in the upper portion of the concrete footing. For such calculations the formulas that follow will be found convenient. These formulas are based on the results of tests made on reinforced concrete under transverse stress, and are sufficiently accurate for all practical purposes.

21. The formula for finding the area of steel reinforcement required, is expressed as follows:

$$A = \frac{W x}{27{,}000\, t}, \qquad (1)$$

in which A = sectional area of steel reinforcement, in square inches, required for each linear foot of spread footings along one side;

W = total load, in pounds, on a portion of projection of footing 1 foot in width;

x = projection of footing, in inches;

t = distance, in inches, from center of action of steel reinforcement to top surface of concrete footing.

The following formula is used to determine the compression created on the concrete due to bending stress:

$$C = \frac{W x}{4.59\, t^2} \qquad (2)$$

The value C is the compression, in pounds per square inch of section, and the other values are the same as in formula **1.** The several values may be more clearly understood by referring to Fig. 16. In this illustration, the portion of the figure that is shaded represents a section of the projection 1 foot in width. The value A in formula **1** is the total sectional area of the two rods marked a. The value W in formulas **1** and **2** is the total pressure on the soil on the shaded area, as shown by the arrows w, while the values x and t are clearly illustrated by the dimension lines thus marked.

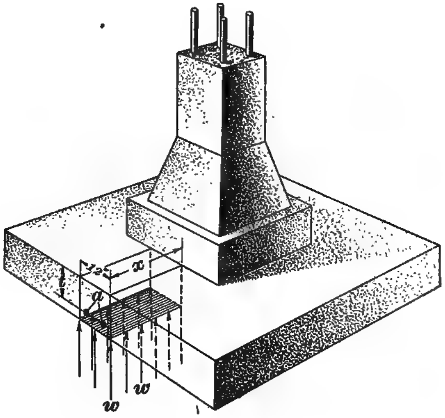


FIG. 16

22. In order to explain the application of the preceding formulas, assume that the reinforced-concrete footing of a column is required to sustain 900,000 pounds, and that, as the soil will safely support 4,000 pounds per square foot of

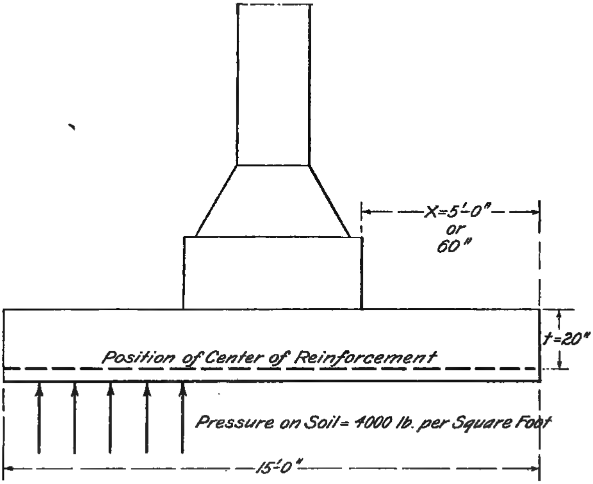

X=5'-0"
or
60"
t=20"
Position of Center of Reinforcement
Pressure on Soil = 4000 lb. per Square Foot
15'-0"

FIG. 17

bearing area, the footing will be 900,000 ÷ 4,000 = 225 square feet. To obtain this area, the footing will have to be 15 feet square, and the design shown in Fig. 17 may be assumed in applying the formula.

In this figure, the projection of the footing, or the distance x, is 60 inches, and the dimension t, or the distance from the center of action of the steel reinforcement to the top of the footing, is 20 inches. The pressure on a projection of the footing 1 foot in width is equal to the unit pressure on the soil, or, $4{,}000 \times 5 = 20{,}000$ pounds, which is the value of w in the formula. These values may be substituted in formula **1** for determining the amount of the steel reinforcement, so that

$$A = \frac{20{,}000 \times 60}{27{,}000 \times 20} = 2.2 \text{ square inches}$$

This area of steel is to be included in each linear foot of the footing course; therefore, the reinforcement may consist of 1-inch square twisted bars spaced practically every 6 inches both ways, or other bars at a spacing to give this sectional area may be used.

The amount of the steel reinforcement having thus been determined, it remains to find out whether or not the concrete in the footing course is overstressed, and formula **2** may be applied as follows:

$$C = \frac{20{,}000 \times 60}{4.59 \times 20 \times 20} = 653 \text{ pounds}$$

This result, which is the maximum compression, in pounds per square inch, on the concrete section, is only slightly greater than the allowable stress of 600 pounds. Therefore, the footing as reinforced and designed may be considered safe, though, in fact, the thickness of the concrete footing might be increased several inches.

EXAMPLES FOR PRACTICE

1. Referring to Fig. 18, what will be the size of the reinforcing rods if square twisted bars are used? Ans. 1.162, say $1\frac{3}{16}$, in.

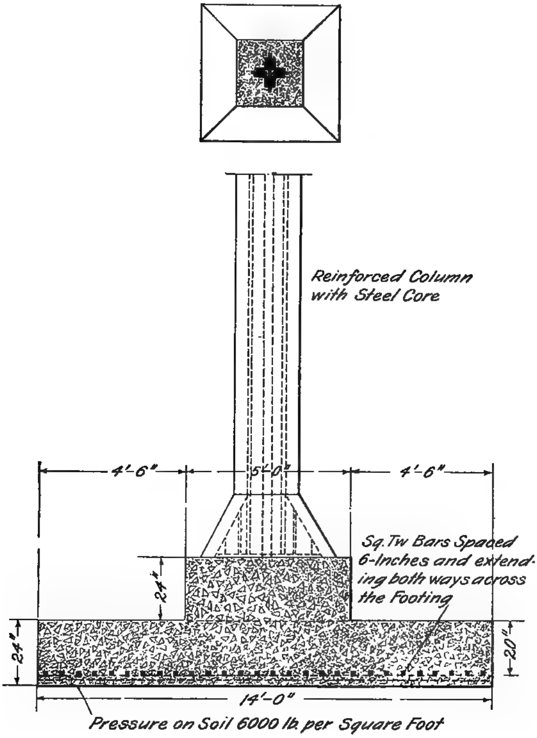

Reinforced Column with Steel Core
4'-6"
5'-0"
4'-6"
Sq. Tw Bars Spaced 6-Inches and extending both ways across the Footing
24"
24"
20"
14'-0"
Pressure on Soil 6000 lb. per Square Foot

FIG. 18

2. Referring to Fig. 18, what will be the maximum compression on the concrete of the projection due to bending stress?

Ans. 794 lb., which is too high

3. What will be the spacing of the reinforcement in Fig. 18 if round rods having the same diameter as the sides of the square bars in example 1 are used? Ans. 5 in., about

REINFORCED-CONCRETE CANTILEVER FOUNDATIONS

THEORETICAL DESIGN

NECESSITY OF CANTILEVER FOUNDATIONS

1. In reinforced-concrete construction, as well as in other types of construction, it is frequently necessary to place a new building close against the walls of an adjoining building. In many instances the wall of the adjoining property rests entirely on its own lot and is not a party wall built half on each side of the party line; also, the adjoining building may be of inferior construction or may be occupied by tenants engaged in manufacture. Under such conditions it is undesirable to tear out the wall and build a party wall.

With buildings of ordinary height and load, provided the basement floor of the new building does not extend below that of the old, few difficulties are encountered in the design of the foundations for the new structure; but if the new building is to be many stories in height and requires extensive foundations along the wall lines, the problem of the design of the reinforced-concrete foundations becomes more complicated, because it is desirable to proportion the footings so that the center of action of the loads will coincide with the center of action from the pressure of the soil beneath. In order to accomplish this desired result in a steel structure, a cantilever-girder system of foundation construction would be employed, as explained in the Section on *Heavy Foundations*, and a similar system can be constructed in reinforced concrete.

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TYPES OF CONSTRUCTION

2. The problem of cantilever-foundation construction in reinforced concrete may be solved by any of the three methods illustrated in Fig. 1. The choice of the method depends on the conditions and requirements of the building structure.

3. The construction shown in Fig. 1 (*a*) is not exactly a cantilever foundation, yet it is very similar to one and is used for the same purpose. It consists of reinforced-concrete beams and columns, as shown, carrying brick curtain walls. Under each basement column is a concrete footing. The basement column is placed directly under the columns of the stories above, but the footing cannot be placed under the basement column, because it would project on the adjoining property. The reinforcing rods of the basement column extend into the footing, so that the column and footing will act as one piece. If the pressure on the soil is uniform, the center of pressure will be at *d*. The line of action of the load on the column is at *a*. These two lines do not coincide. There is therefore a bending moment in the basement column equal to the load on the footing multiplied by the distance between the lines of action of *d* and *a*. The basement columns must therefore be reinforced with additional steel on the outside to resist this bending moment. This same moment is also transmitted from the column to the first-floor girder *c*, and produces in this beam a negative moment that tends to neutralize the positive moment due to the floor loads. The basement column must therefore be carefully tied to the beam *c* with steel reinforcement. Thus, the basement column is held rigidly at both ends.

The curtain wall in the basement carries no load except its own weight, and rests on its own separate foundation between columns not shown in the illustration.

4. Another method of solving the problem of cantilever-foundation construction in reinforced concrete is illustrated in Fig. 1 (*b*). Here, the construction is used in much the same manner as steel construction would be employed for cantilever foundations and calculations similar to those explained in *Heavy Foundations* are made.

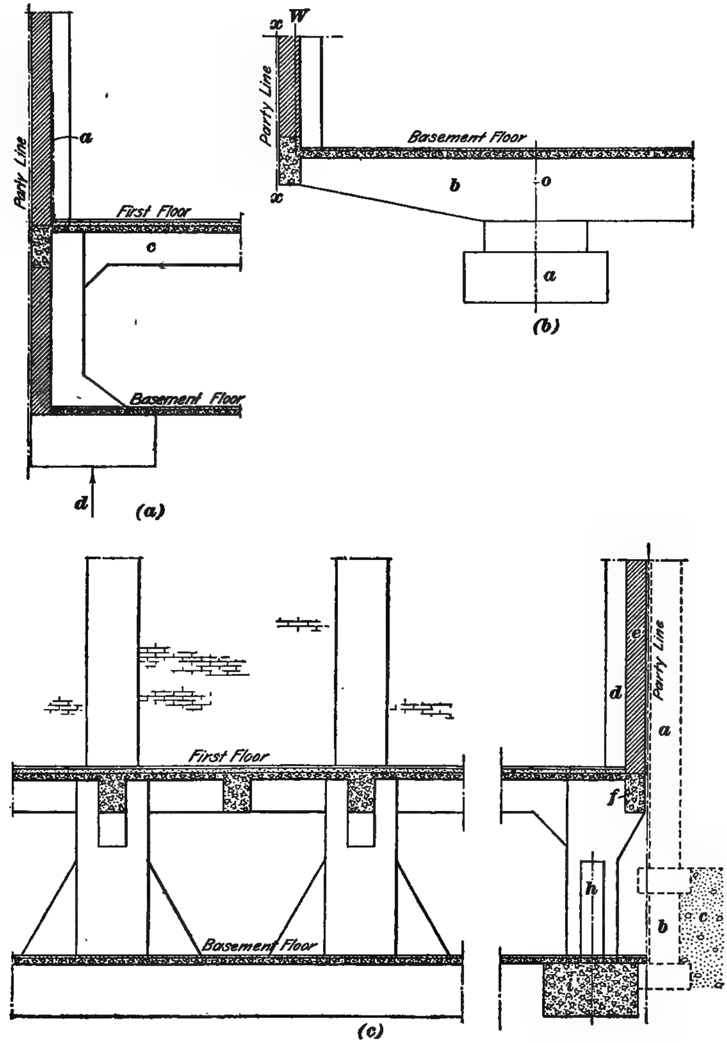


Fig. 1

In this type of construction, the footing *a* is some distance within the party line *x x*, and if sufficient area cannot be secured by the adoption of isolated-pier construction, then these footings can be extended the full length of the building and reinforced for the bending moment between cantilevers. The weight *W* tends to create a bending moment about the point *o*, causing tensile stresses in the upper part of the beam *b*.

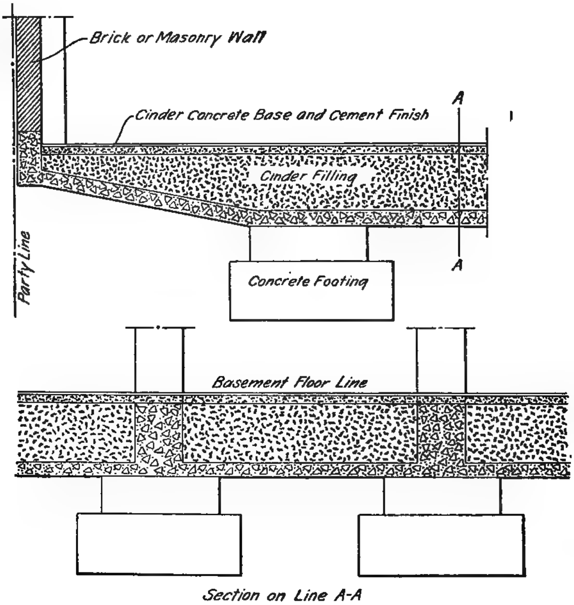


FIG. 2

If it is necessary to obtain compressive resistance at the bottom of the cantilever and throughout the length of the beam between supports, a slab construction can be adopted in conjunction with the cantilever beam. This may be used to carry the basement floor by filling in between with cinders and introducing the regular cement floor, as shown in Fig. 2.

5. In Fig. 1 (*c*) are indicated the conditions that require a typical cantilever-foundation construction in reinforced concrete. At *a* is shown the wall of the old building. The cellar bottom of this building is at a considerable distance above the cellar bottom of the new building. It is quite evident, therefore, that the wall *a* must be underpinned, as at *b*. In this way, the danger of the bank *c* giving in during the process of excavating for the new foundation is avoided.

The cantilever shown in the figure supports the new building, which is constructed with concrete wall columns extending the full height of the building. These concrete wall columns are shown at *d*, and between them are built brick curtain, or enclosure, walls 12 or 13 inches in thickness, as at *e*, the curtain walls being supported at each story of the building by means of lintel beams, as at *f*. Often, the soil under the building is such as to require a continuous footing, thus making it necessary to distribute the column loads.

In order to transmit the load from the wall columns over the width of the continuous footing, the heavy concrete basement column is often supplied with an inverted bracket or web, as at *h*. The continuous footing *i* must also be so proportioned and reinforced that it will be able to sustain the several column loads in the same way as a continuous beam. This ordinarily requires reinforcement at the top and the bottom of the footing.

STRESSES PRODUCED IN REINFORCED-CONCRETE CANTILEVER FOUNDATIONS

GENERAL CONDITIONS

6. The type of reinforced-concrete cantilever foundation construction most commonly employed is that shown in Fig. 1 (*c*), so that particular attention will be given to the action of the stresses in this construction. In order to understand these stresses, reference is had to Fig. 3 (*a*). Neglecting the load of the first floor, it may be said that the cantilever beam *b* is held in equilibrium by three forces, namely, the load W from the wall columns, the reaction R_1 from the basement columns, and the load R_2 of the interior column.

The greatest bending moment occurs on the line $y\,y$, and is equal to $W\,x$. The beam b is a cantilever, and, being subjected to negative bending moment, it must be reinforced at the top. The load on the first floor will somewhat reduce this moment. The amount of this reduction of negative bending moment will depend on the amount of live load acting at the time. The maximum negative moment at any point in the beam b is therefore equal to the negative moment at that point caused by the cantilever action, less the positive moment caused by the dead load of the beams and floor slab.

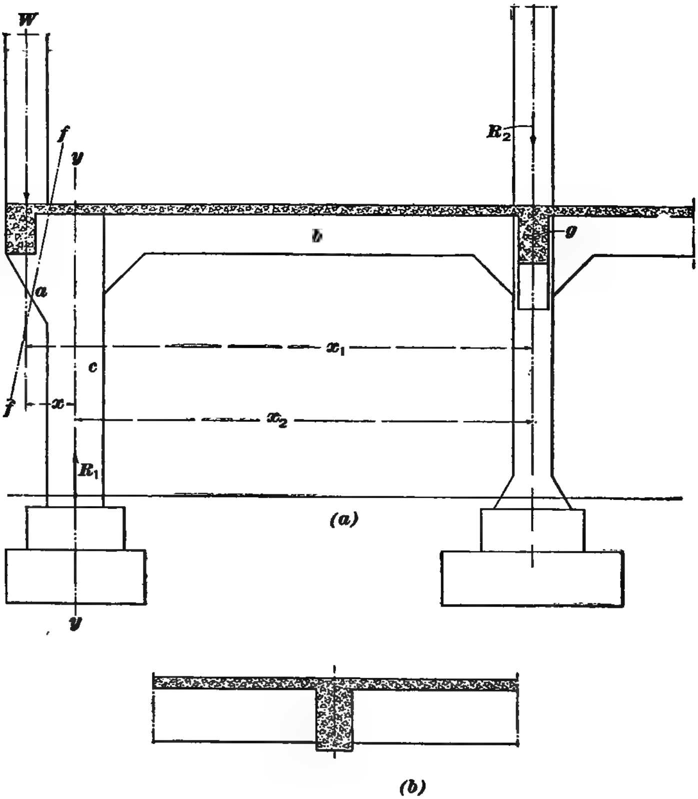


FIG. 3

The minimum negative moment at any point is the negative moment caused by the cantilever action, less the positive moment caused by both the dead and live first-floor loads. It can be seen that if the floor loads that cause the positive moment are sufficiently large, the total moment may be positive, or at least vary between negative and positive as the live load varies. Great care must therefore be exercised to investigate carefully the bending moment at various points in the beam. The steel should then be placed in its correct position, either at the top or the bottom of the beam or at both the top and the bottom.

7. The easiest method of designing the beam is to plot the bending-moment diagram, first for the cantilever effect and then for the moment due to the first-floor load, using the method given in *Forces Acting on Beams*. These two diagrams, ordinate by ordinate, should then be subtracted from each other and the resultant diagram obtained. It will be found that the bending moment varies greatly along the length of the beam and is greater near the column *c*. The steel reinforcement in the beam *b* near the column *c* may be reduced in the other end of the beam by stopping off the rods at various points along its length.

If the beam *b*, instead of joining the girder *g* directly at the column, joins it in the middle of its span, as shown in (*b*), the conditions are somewhat different. Although the stresses in the beam *b* are exactly the same as before, the girder *g* is now affected. The end of the beam *b*, being no longer held down by the load on the central column, presses up on the girder *g* with a force R_2. This causes negative bending moment in the girder. The first-floor load causes positive bending moment in the girder. These two moments must be combined in the same manner as was suggested in regard to the beam *b* to find the resultant moment, which may be either positive or negative.

8. The projecting end of the cantilever beams, as at *a*, must be adequately reinforced to resist the tension and compression stress in the top and bottom of any section through it. Its section must likewise be examined to determine whether the shearing stresses, as along the line *f f*, are adequately resisted by the rod reinforcement together with the shearing resistance of the concrete.

The column *c* forming the support of the cantilever must resist the reaction R_1, which is greater than the weight *W*, because the lever arm x_2 is smaller than the lever arm x_1, which is the leverage through which the weight *W* acts about the center column. The column should therefore be well reinforced.

Owing to the fact that the concrete in the bracket is likely to shear off, as indicated by the line *f f*, numerous stirrups should be introduced. These stirrups should extend across this line of possible shear, so as to tie in the concrete of the bracket and to provide against any such possible chance of failure. In all instances, the reinforcement of the girder *b* should have ample stirrups running through the mass of this structural member. However, in the placing of stirrups and light or secondary reinforcement, the matter of filling the forms should be considered; also, the reinforcement should be arranged so that there will be an ample body of concrete around it and that there will be sufficient space between the several bars and rods to make a solid concrete structure when completed.

SYMMETRICAL DESIGN

9. There is another way of solving the above problem if the building is symmetrical as regards its construction on both sides; that is, when the building is located in the center of a city block between adjoining buildings, the conditions at each party line being theoretically the same. This method of solution is described by means of Fig. 4.

In this figure the building is supported on the wall footings, as indicated at *a* and *b*, and on an intermediate interior row of column footings, as at *c*. The cantilevers d_1 and d_2 sustain the exterior wall loads. It is quite evident that the weights W_1 and W_2 tend to rotate about the points c_1 and c_2 of the wall footings, and also that equilibrium of rotation is secured by the reactions R_1 and R_2 and the rigidity of the girders d_1 and d_2. The latter rigidity can be secured by placing sufficient reinforcement in such a position as to resist tensile stresses.

The solution is simple, as d_1 and d_2 may be considered as separate beams. If there is any likelihood of the reactions of d_1 and d_2 being greater than the force P, that is, of lifting up the central column, they may be considered as one long beam with a central load and reinforced as such. In any case, the bending moments may be calculated by the rules given in *Forces Acting on Beams.*

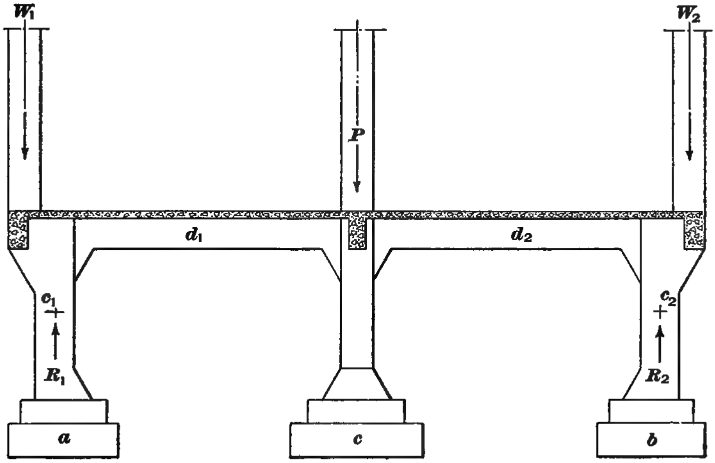


FIG. 4

CONTINUOUS FOOTINGS

10. Frequently, all three types of reinforced-concrete cantilever construction under discussion can be avoided, especially where the soil is excellent, by adopting the construction known as a **continuous footing,** which is shown in Fig. 5. In this construction, the necessary bearing area on the soil is obtained by extending a longitudinal footing from column to column that has sufficient depth and is so reinforced as to transmit the load upon the soil. In this way area is gained by increasing the longitudinal dimension of the footing rather than the width. In adopting this construction, it is considered advisable by some engineers to cut clear through the concrete footing at the center of its length between each two columns, thus insuring that each column will rest solidly on the soil under it and that, therefore, each section of footing will carry its calculated load. This is suggested because there may be soft places in the soil of such an extent that it may be impossible to determine the necessary reinforcement required to span such spaces. It is almost certain, however, that where this type of construction is well designed, no such precaution need be taken, especially if reinforcement is provided in both the top and the bottom

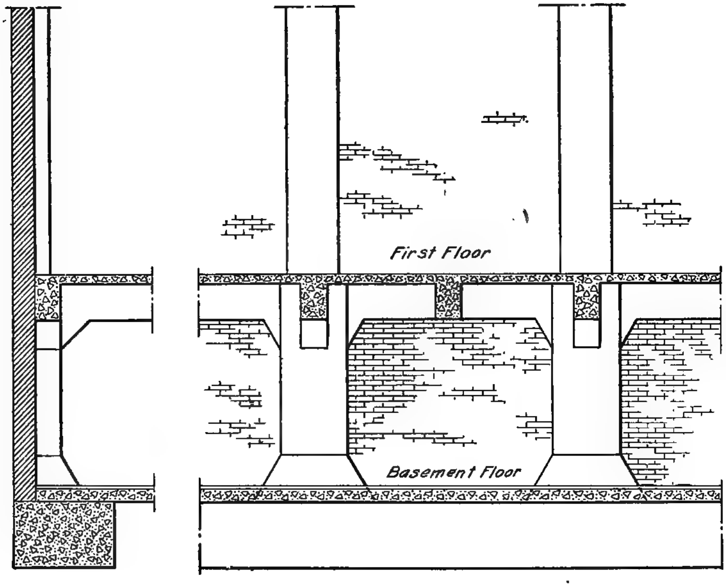
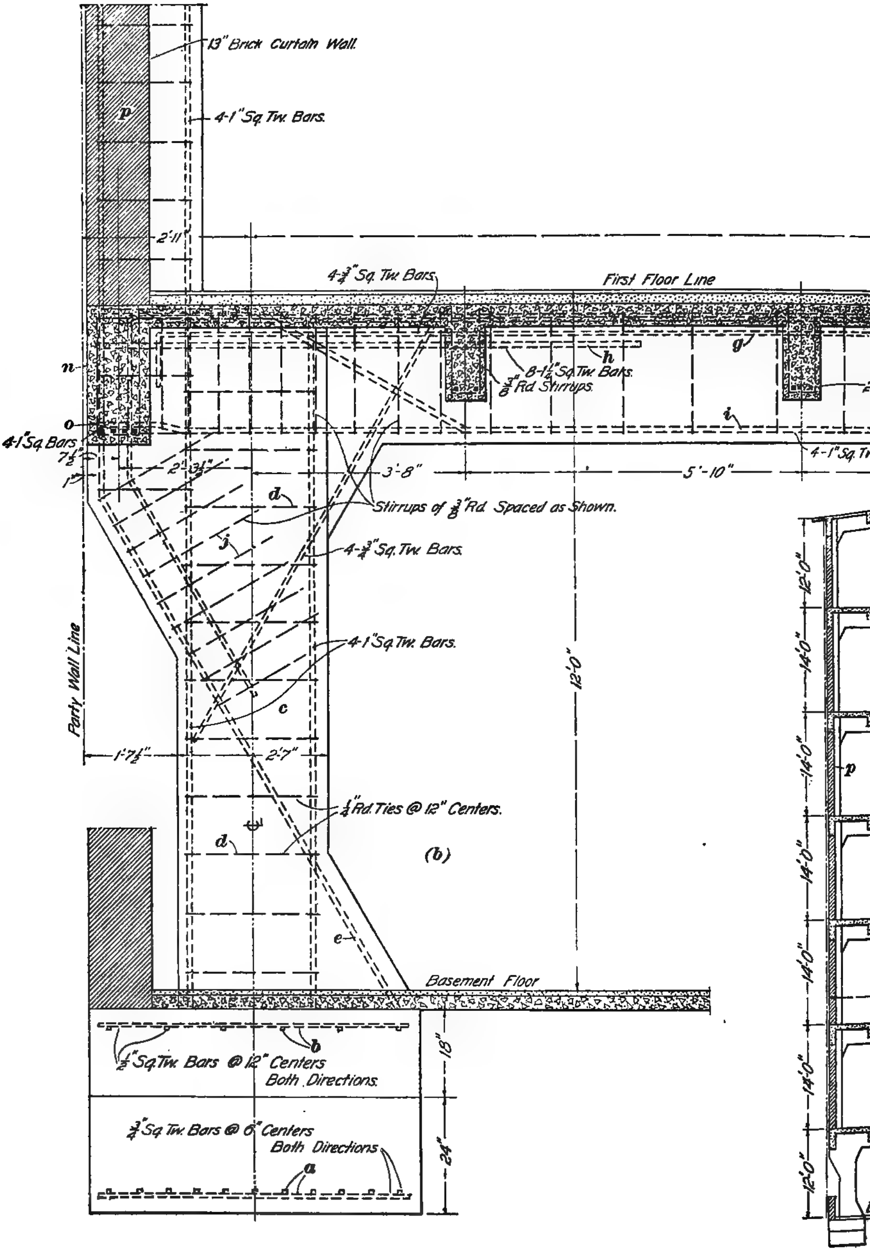
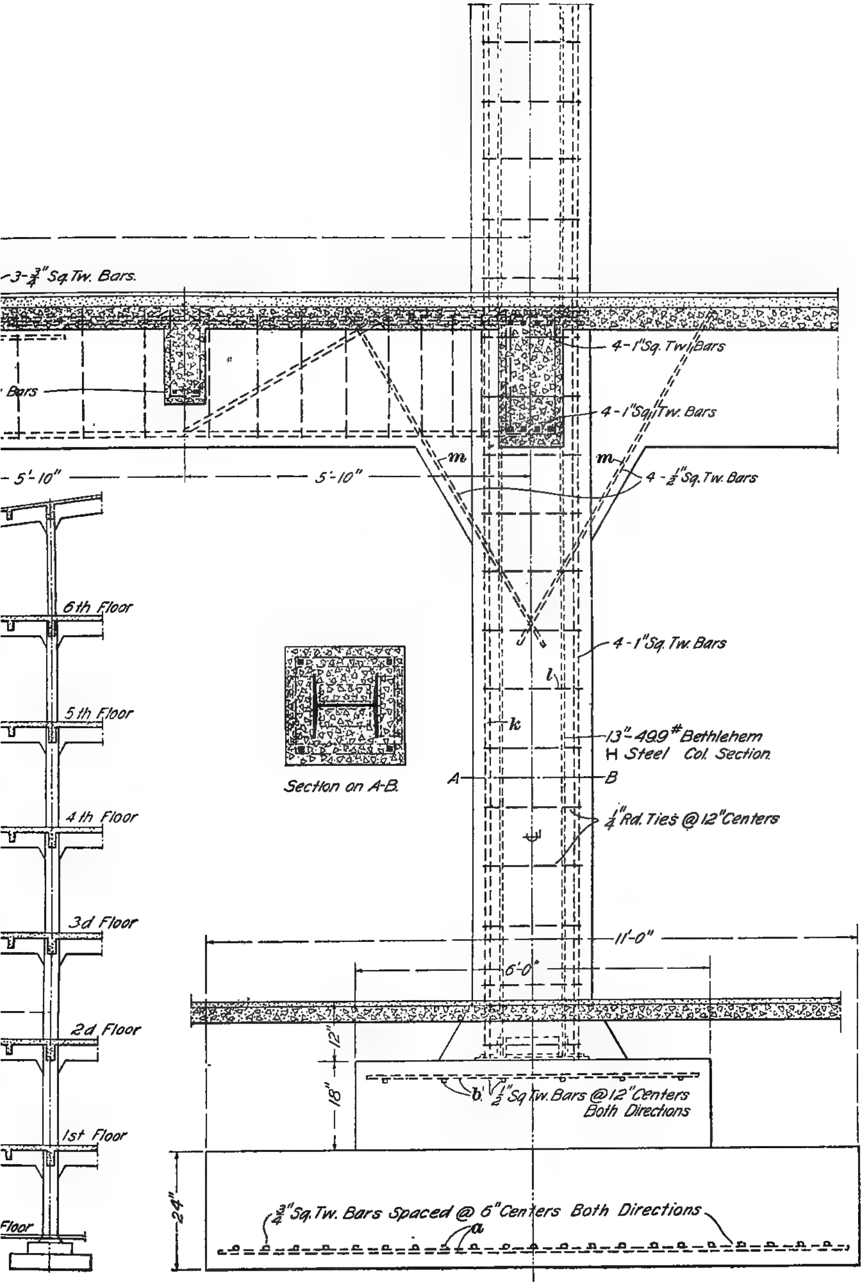


FIG. 5


13" Brick Curtain Wall
p
4-1" Sq. Tw. Bars.
2'-11"
4-3/4" Sq. Tw. Bars
First Floor Line
n
o
4-1" Sq. Bars
7 1/2"
1"
h
g
8-1 1/4" Sq. Tw. Bars.
3/8" Rd. Stirrups.
i
4-1" Sq. Tw.
2'-3 1/2"
3'-8"
5'-10"
d
Stirrups of 3/8" Rd. Spaced as Shown.
j
4-3/4" Sq. Tw. Bars.
4-1" Sq. Tw. Bars.
c
12'-0"
Party Wall Line
1'-7 1/2"
2'-7"
1/4" Rd. Ties @ 12" Centers.
d
(b)
e
Basement Floor
b
1/2" Sq. Tw. Bars @ 12" Centers Both Directions.
18"
3/4" Sq. Tw. Bars @ 6" Centers Both Directions.
a
24"
12'-0"
14'-0"
14'-0"
p
14'-0"
14'-0"
14'-0"
12'-0"


3-¾" Sq. Tw. Bars.
Bars
4-1" Sq. Tw. Bars
4-1" Sq. Tw. Bars
m
m
4-½" Sq. Tw. Bars
5'-10"
5'-10"
6th Floor
5th Floor
4th Floor
3d Floor
2d Floor
1st Floor
Floor
4-1" Sq. Tw. Bars
l
k
13"-49.9# Bethlehem
H Steel Col. Section
A
B
Section on A-B.
¼" Rd. Ties @ 12" Centers
11'-0"
6'-0"
12"
18"
b
½" Sq. Tw. Bars @ 12" Centers
Both Directions
24"
¾" Sq. Tw. Bars Spaced @ 6" Centers Both Directions
a

of the rectangular concrete beam that forms the footing and transmits the loads from the column to the soil.

In the design of all such special foundation construction, more than the ordinary amount of care and attention is required in order to reinforce the concrete successfully and in an economical manner.

PRACTICAL DESIGN

11. In Fig. 6 is shown the detail drawings of a reinforced-concrete cantilever-foundation construction for a six-story building, the floors of which are designed for light manufacturing purposes. In view (*a*) is shown a diagrammatic cross-section of the building, which illustrates the conditions of loading and the spans of the cantilever and other girders. The live load to be supported by the floor construction and the cantilever girder is 220 pounds per square foot, and the soil beneath the footings is capable of supporting safely 6,000 pounds to the square foot.

The distance between bents, or the distance center to center, of the reinforced-concrete wall columns is 8 feet, so that when the weight of the curtain wall *p* is added to the total dead and live load of 320 pounds for each floor and the total roof load of 120 pounds, the total load on each wall column that is transmitted to the end of the cantilever beams is almost 300,000 pounds. This load, together with the weight of the construction of the first floor, determines the size of the footings, and is the basis for the calculation to determine the size of the concrete cantilever girder, its reinforcement, and the size of the concrete column or pier supporting it.

12. The details of the construction and the reinforcement are shown in Fig. 6 (*b*). The footing is reinforced against failure from tranverse stress by bars *a*, placed near the bottom of the footing, and further by a mattress of rods or bars placed directly beneath the bearing of the foundation wall column, as at *b*. The column, or wall pier, *c* is reinforced with vertical reinforcing bars, which are tied at close intervals with wire or loop ties *d*. Particular attention is called to the arrangement of the vertical column rods and the manner in which the outside rods of the foundation pier or column run straight through and up into the column above, being continued as the inside rods of the upper tiers. The outside rods of the first-floor wall column are bent so as to pass obliquely through the cantilever bracket of the foundation pier or column and the flare at the bottom, as shown at *e*. By arranging these rods in this manner the end of the cantilever is greatly strengthened to withstand compression through the lower part of its section; besides, these rods help to resist oblique shearing stresses through the end of the cantilever and the foundation column.

13. The reinforcing rods of the cantilever girder are arranged as shown at *f*, *g*, *h*, and *i*. The rods in the upper part of the girder are arranged in three rows, or layers, and are stopped off in length in practically the same manner as the flange plates of a built-up girder, because the bending moment is reduced toward the interior column. All the rods extend through to the end of the cantilever, the rods *f* and *g* being bent downwards at the end, so as to make a secure bond in the concrete and to give additional strength to the brackets in shearing resistance and in compression from the transverse, or bending, stress. Only one set of the reinforcing rods, as at *f*, extend entirely through into the interior column, because the bending moment is so reduced that this set is all that is required. The bottom rods, as at *i*, are inserted to improve the resistance to compression in the cantilever girder beam at the bottom.

Both the girder and the cantilever bracket are well supplied with stirrups properly arranged to resist any horizontal or oblique shearing stresses, as shown at *j*. The vertical reinforcement of the interior column is shown at *k*, and the ties and bracket reinforcement, at *l* and *m*, respectively. The lintels, or connecting beams, at the end of the cantilever are shown at *n*. These beams have the advantage of the added strength in compression imparted to them by the reinforced-

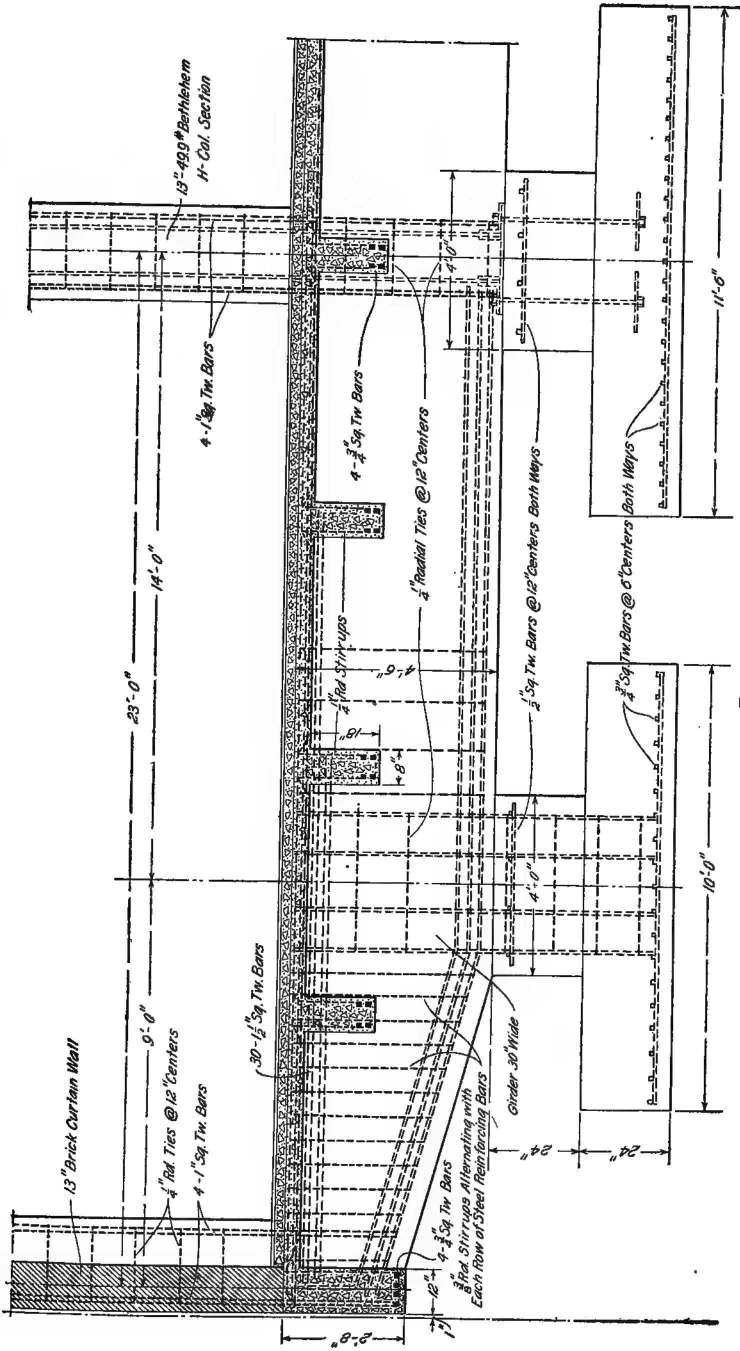

13"-49.9# Bethlehem H-Col. Section
4-1" Sq. Tw. Bars
4-3/4" Sq. Tw. Bars
1/4" Radial Ties @ 12" Centers
1/2" Sq. Tw. Bars @ 12" Centers Both Ways
3/4" Sq. Tw. Bars @ 6" Centers Both Ways
4'-0"
11'-6"
23'-0"
14'-0"
1/4" Rd. Stirrups
4'-6"
18"
8"
4'-0"
10'-0"
30-1 1/2" Sq. Tw. Bars
9'-0"
13" Brick Curtain Wall
1/4" Rd. Ties @ 12" Centers
4-1" Sq. Tw. Bars
4-3/4" Sq. Tw. Bars
3/8" Rd. Stirrups Alternating with Each Row of Steel Reinforcing Bars
Girder 30" Wide
24"
24"
12"
1"
2'-8"

Fig. 7

concrete slab. The steel reinforcement of these beams is shown at *o*. Some of these rods are bent upwards toward the ends of the lintel beams and well lapped over the top of the cantilever brackets, so as to give the beams the additional strength of a beam with the ends well fixed, or secured.

14. In Fig. 7 is shown a reinforced-concrete cantilever-foundation construction designed to carry the same load on the end of the cantilever as was required of the construction illustrated in Fig. 6. The several dimensions show the distance of the footings within the walls, and as the cantilever girders have a considerable projection beyond the foundation footings, they must be very strongly reinforced in both the top and the bottom, as shown. The several details of this construction are worked out in the illustration, and an analysis of these will show that the several principal and secondary stresses created in the structure are amply provided for.

FORM WORK

FORMS FOR WALLS, SLABS, AND BEAMS

INTRODUCTION

1. In the erection of reinforced-concrete work, nothing requires more careful consideration than the construction of the **form work,** or molds, necessary to shape and support the concrete until it has thoroughly set and hardened. Throughout the practice of reinforced-concrete construction various methods of form constructions are in use. Generally, a particular type of form work will be found to be confined to a locality, or else will be used by a single firm of reinforced-concrete contractors, the advocates of the several systems believing the system that they use to be the best and most economical.

2. Supervision of Form Construction.—While the construction of the forms is frequently left entirely to the contractor, the architect or the engineer will do well to exercise a supervision over it; besides, the specifications for reinforced-concrete work should be written so as to outline in a general way the manner in which the forms are to be constructed. In all instances, the architect or the engineer should control the time for the removal of the forms, and should be capable of insuring the safety of the structure by intelligent inspection of the false work and supports.

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The many types of forms used in reinforced-concrete construction, as well as their general requirements, will be explained in the following articles, and an effort will be made to point out the typical features of each type of construction, with the advantages and disadvantages of each. On the correct design and construction of the forms for reinforced-concrete work depend the safety, economy, and appearance of the entire structure.

3. **Economy in Form Construction.**—As a great amount of material and labor is required in the construction of forms, it is evident that the cost of this work is not a small part of the cost of the reinforced-concrete structure, and by the exercise of care in the design, material and labor can be saved.

The greatest economy is gained by constructing the forms so that they can be used over and over again in the structure. It is not unusual to construct a ten-story building by using over again the form work employed for the first three stories. This fact should be borne in mind in the design of reinforced-concrete buildings, because it is frequently cheaper to keep the columns of the same dimensions throughout in order to save the cost of changing the forms; also, for the same reason, it is sometimes advisable to use beams and girders of the same thickness and depth on the several floors, even if they are designed for lighter loads.

Economy in construction can be gained by fastening the form work together with a minimum amount of nailing. Every nail that is driven gives trouble when the forms are taken down to be replaced for the upper floors. In many constructions, wedges and clamps are used instead of nails or screws if the forms are to be reused.

4. **Safety of Form Work.**—The safety of a reinforced-concrete floor, as well as that of the entire structure, may be jeopardized by faulty form construction. Many of the failures of reinforced-concrete construction can be traced directly to weakness in the forms, due to faulty construction of their supports or the false work. For instance, if the forms

for beams and girders should be supported by struts of insufficient strength, causing a collapse of the form in one part, the probability is that the forms of the whole floor would collapse with it and deposit their contents on the floor below. A load suddenly applied in this manner to the floor below would very likely cause the collapse of the whole building. The same result would follow a failure of column forms. If the supports are such as to allow the sagging of a girder form, it would displace the reinforcing rods, giving them a location higher up in the girder. This would decrease the strength of the girder and might subsequently jeopardize the safety of the building.

In order that there may be no doubt regarding the safety of the form work, it should be carefully inspected before any of the concrete is placed. It should be seen to that the upright supports and braces are properly placed and are sufficiently strong to carry the weight of the wet concrete, for, frequently, the false work and braces are improperly placed and do not have sufficient bearing. It is also not uncommon to find reinforced-concrete forms braced against green masonry and brickwork and struts secured with a few wire nails, whereas they should be notched into the supporting timbers and well spiked.

5. Effect of Form on Appearance.—By the use of well-constructed forms, not only are economy and safety attained, but the appearance of the work is greatly improved as well. Form work is so carelessly constructed at times, that the ends of the upright supports, or props, rest on soft earth in such a manner as to allow the beams and girders to sag under the weight of the wet concrete. The concrete sets with this sag in the beam and girder members, and the entire floor system looks as if it were greatly deflected. Then, too, the sides of the beam and girder forms, when these members are to be deep, are sometimes so poorly braced and constructed as to bulge or warp. This causes these members to be badly twisted and thus spoils the appearance of the structure.

In all cases, care in the construction of the form work shows materially in the appearance of the finished work. It is usually found that the care and judgment displayed in the construction of the form work is a criterion of the entire construction, and that the results of care in this work will insure a safe building and a fine clean appearance.

6. Materials Used in Form Work.—Most form work is constructed on the site of the operation from lumber purchased especially for the purpose, although at times the beam, girder, and column forms are made at the mill and sent to the operation ready for erecting in place. On account of the cost of form construction, many attempts have been made to make adjustable forms that can be used repeatedly for any work. For this purpose, both wood and sheet metal have been employed with some success.

CONSTRUCTION AND FINISH OF FORM WORK

7. Timber Used in Form Construction.—The kind of timber to use in the construction of forms for reinforced-concrete work depends to a considerable extent on the available supply in the particular locality in which the work is to be erected. It also depends on whether the material after its first use is to be again employed in similar work, and whether the building is to have many floors of the same design and construction or whether it is to be only two or three stories in height. Ordinarily, good material should be used in form construction, for by its use is secured smooth and true work.

For the support of the form work, it is the custom to use as cheap a grade of rough, sound lumber as can be purchased. For the form work, or that portion of the work in contact with the concrete, it is best to use a good grade of well-seasoned yellow or hard pine, because this material possesses both strength and grain of sufficient closeness to prevent badly splintered form boards. As a rule, spruce and hemlock cannot be successfully employed in the construction of form boards. These materials are liable to split in nailing and to curl on account of the moisture from the wet concrete.

8. Sizes of Timber Used in Form Construction. If the forms are to be used only one time, they are usually made up of 1-inch dressed material; whereas, if they are to be used several times, the side and bottom form boards of the beams and girders are made of either 1½- or 2-inch dressed planks, because these usually withstand the successive operations of demolishing and erecting much better than the lighter material. The shores and supports for the centering generally consist of rough 3″×4″ hemlock studding, it having been found by practice that such studs, when placed from 4 to 6 feet on centers, possess the necessary strength to carry the weight of the wet concrete for usual floor systems.

For ledger boards, cross-braces, and ties, as well as for nailing strips and battens, 1″×6″ or 1½″×6″ sawed boards are found convenient; 2″×6″ and 3″×6″ joists are used to a considerable extent for the support of slab centering and for the bracing of the supports. Timbers as large as 4 in. × 6 in., 6 in. × 8 in., and 8 in. × 10 in. are used to brace and secure the form work for heavy concrete-wall and buttress construction, and all sizes of timbers are used for the false work of concrete-arch construction.

9. Metal Used in Form Construction.—Sheet metal has been used to some extent in the construction of forms. Frequently, molds, or forms, for the construction of architectural ornament or detail are made of either galvanized sheet iron or wood lined with this material. If very smooth concrete work is desired, the wooden forms are sometimes covered with galvanized iron. This material is admirably adapted to the construction of the forms of cylindrical columns, for by the use of sheet metal properly banded, the column presents a smooth appearance upon completion—an effect practically impossible to obtain by using a column form built up of wooden staves. Generally, No. 27 gauge galvanized iron is used for the lining of forms.

Black sheet iron has been used to a limited extent for molding concrete work. The concrete, however, adheres more strongly to black plate than to galvanized iron, and, besides, the plate is liable to rust and stain the work. In building concrete stacks, sheet iron about $\frac{3}{16}$ inch in thickness is generally used for the forms.

10. Coating of Forms.—In some instances, both wooden and metal forms for reinforced-concrete work are coated on the side next to the concrete in order that the forms may be detached more readily from the concrete work after it has set. Coating the forms also serves to prevent the marking of the grain of the wooden forms on the finished concrete work.

Dead oil, or crude petroleum, has been used with success for this purpose; it is probably as efficient and as economical as any oil or coating that can be used. It is not unusual to soap wooden forms, and in some cases tallow and bacon fat have been employed. The latter is especially recommended for coating metal forms, as it seems to give the best results with forms of this material. In some instances, wooden forms have been covered on the inside with paper, and even canvas has been used, although it is usually found that the paper adheres to the concrete work and is detached only with difficulty.

In one instance the forms were varnished and fine stone dust was blown on the varnished surface before it had dried. These forms were easy to remove, and the markings of the grain of the wood on the cast concrete work were scarcely noticeable.

It is not customary, however, to oil or coat the forms unless they are to be used for fine exterior work.

If lettering is to be formed on the face work, the die should be made as carefully as a pattern for casting metal, and the forms should be shellacked and sandpapered so as to give them a smooth finish and to fill the grain of the wood. In this way they will be prevented from adhering to the concrete. It is very difficult to cast lettering in concrete face work. For this purpose the **V**-sunk architectural lettering offers the least difficulty.

FORMS FOR FLOOR SYSTEMS

11. Common Types of Form Work.—In Fig. 1 is shown a type of form work extensively used for the construction of reinforced-concrete floor systems. In the system for which this form work is arranged, the beams and girders intersect at the columns and the girders support an intermediate floorbeam. In (*a*) is shown a perspective view of the form work partly constructed for a complete panel; in (*b*), an

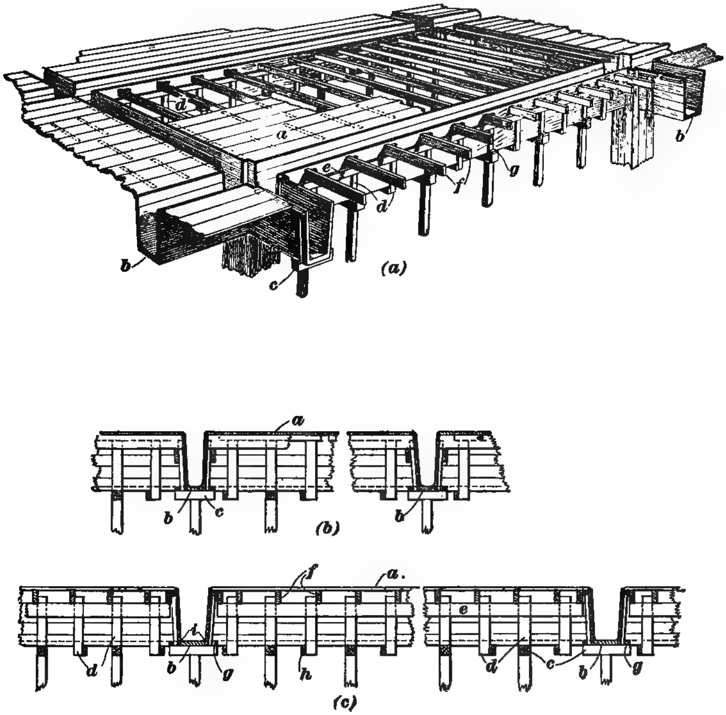


Fig. 1

elevation taking a section through the beam form; and in (*c*), a section through the girder form. A perspective of the forms at the intersection of a beam and girder is illustrated in Fig. 2.

This form work is designed so that light $\frac{7}{8}$-inch dressed tongued-and-grooved material may be used extensively in its construction. It is arranged so that the side forms of the beams and girders, together with the slab form boards, may be removed without the necessity of removing the supports directly underneath the beams and girders.

12. The form work for the slab construction, as shown at *a*, Fig. 1, is also made up of $\frac{7}{8}$-inch dressed tongued-and-grooved boards or planking, well battened together at the ends and at intervals throughout their length. The bottom board *b* for the form work of the beams and girders is of 2-inch dressed plank and sets between the side planking of the forms. The entire form work is supported by means of crosspieces *c*, *c* that rest on 3″×4″ studs. The sides of the forms, like the form work for the slabs, is made up of $\frac{7}{8}$-inch tongued-and-grooved planking. This material runs lengthwise of the forms and is well battened together with 2″×4″ battens *d* placed at intervals of about 18 inches. By this means, the side forms of the beams and girders can be taken away in one piece, and can be used in the construction of the form work for an upper floor. The form work for the slab is supported directly from the side pieces of the beams and girders by means of a ledger board *e* of $1\frac{1}{4}$″×6″ material nailed to the battens of the sides of the forms. The supports of the slab form work consist of 2″×6″ joists *f* placed at intervals of about 18 inches. These joists rest upon the ledger board *e*, and are spiked to the sides of the battens *d* that hold together the planking of the side forms of the beams. The side forms are kept from spreading at the bottom by wedging them between cleats *g* nailed to the top of the crosspiece *c*, and intermediately by a 1″×3″ piece *h*, Figs. 1 and 2, nailed to the ends of the battens that extend below the bottom board of the form.

13. As shown in Figs. 1 and 2, the forms for the beams and girders are tapered considerably. In addition, the beams and girders are chamfered by inserting wooden fillets *i*, Fig. 1 (*c*), in the corners of the form. A finish is given to the intersection of the angle made by the slabs with the beams and girders by chamfering the upper edge of the side forms, as shown at *j*, Fig. 2. By making the beams and girders of the same depth, a design for which this type of form work is particularly suitable, there is some saving in the cost of labor and material, and it frequently simplifies the hanging of shafting or other fixtures of installation.

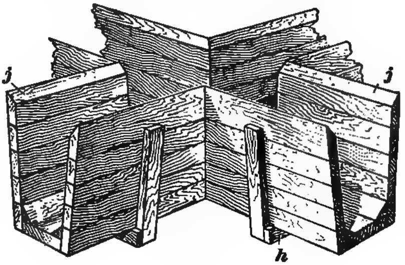


FIG. 2

In this type of form work, only a minimum amount of heavy planking is required; also, the sides of the forms may be removed and placed in one of the upper floors while the main supports under the beams and girders remain in place.

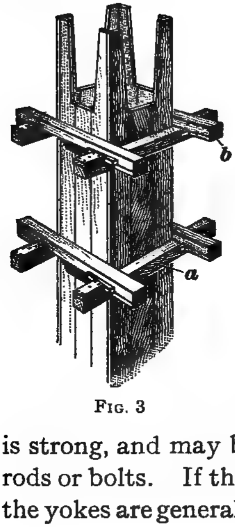


FIG. 3

14. The column form for the type of form-work construction just described is shown in Fig. 3. It is made of either 1- or 1½-inch dressed planking that is run vertically and is well battened. The sides of the forms are prevented from spreading by means of ties, or yokes. These consist of 3″×4″ timbers *a* with cleats *b* 1 inch thick nailed on the ends. By this means, a yoke is formed, locking the ends of the wooden ties where they cross, by the insertion of wooden wedges. A tie of this kind is strong, and may be constructed without the use of threaded rods or bolts. If the column is of a height greater than 3 feet, the yokes are generally spaced about 2 feet from center to center.

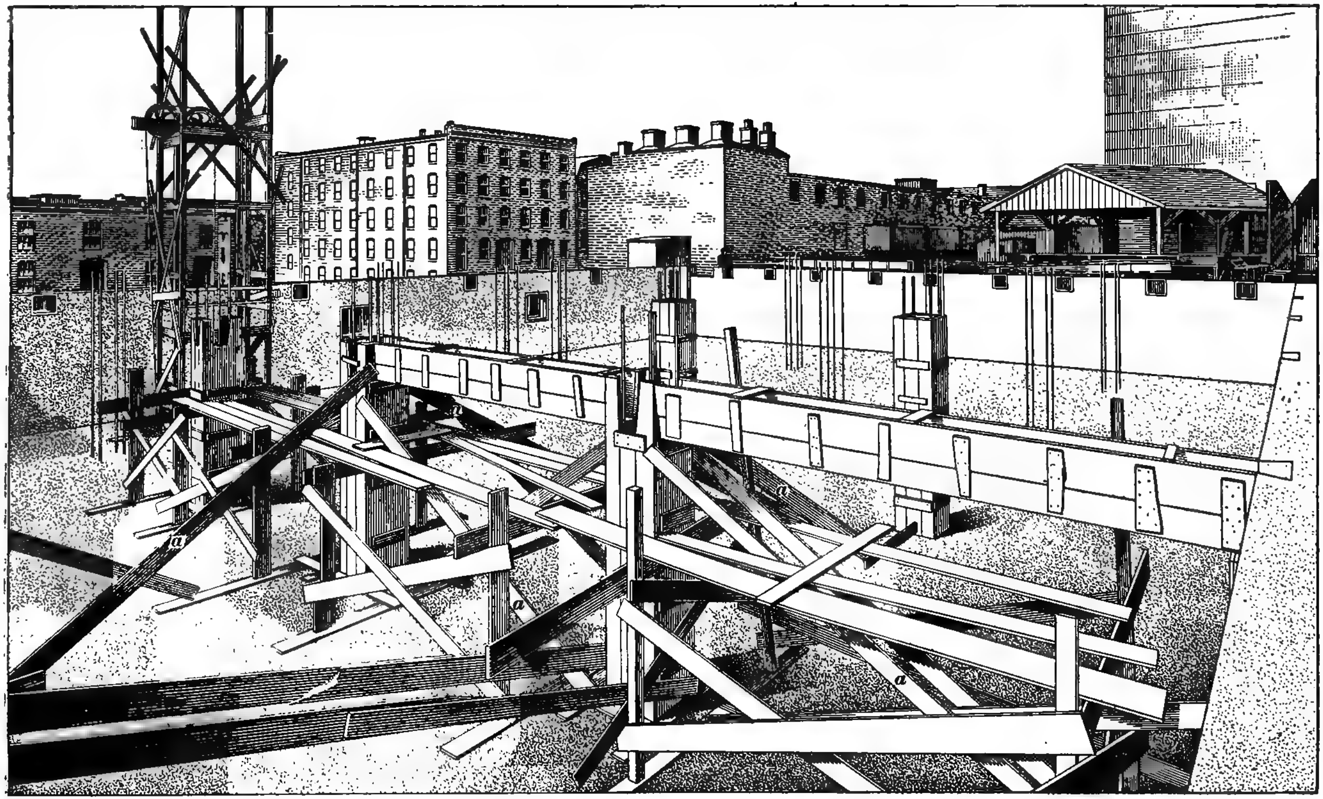


Fig. 4

As will be observed, the sides of the column forms are cut out at the top in order to allow the beam and girder forms to run into the inside face. In this way a support is provided for the forms over the columns.

15. A clear idea of the actual construction of the column forms, the manner in which they support the beam and girder forms, and the way in which they are braced preliminary to the construction of the forms for the floor system may be had by referring to Fig. 4. In this figure, the yokes used to secure the sides of the column forms against failure have not been placed. In bracing the columns, boards 1 in. × 6 in. or 1 in. × 10 in. are run from the top of the column form to the bottom of the nearest column, as shown at *a*.

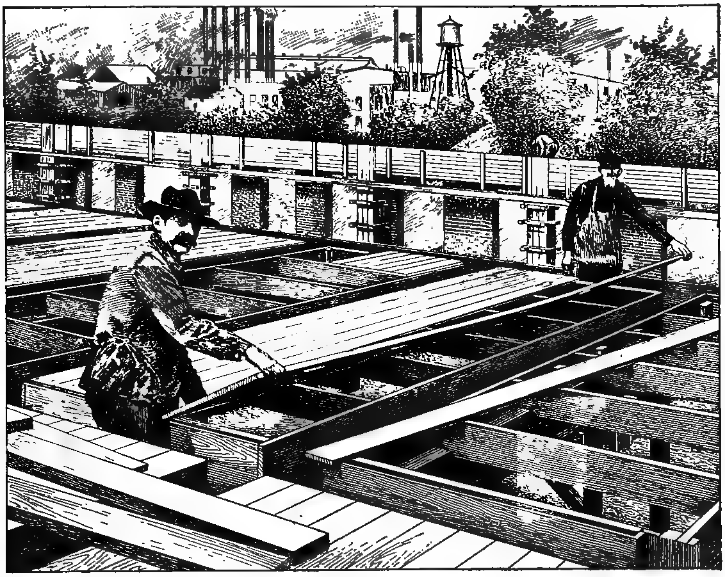


Fig. 5

In Fig. 5 is clearly shown the method of constructing the form work for the slabs. Here, the slab form work is made in halves, so that it can be readily handled and used again for the construction of a top floor. In this figure, the side forms for the beams and girders are in place, and the joists for the support of the form work are ready to receive the form boards for the slab construction.

16. Forms Constructed of Plank.—A superior type of form for a reinforced-concrete floor system is shown in Fig. 6. The wooden forms are supported at the columns by the form construction for these members, and elsewhere throughout the building by 3″×4″ studs *a* that rest upon a footing piece *b*. Over the top of the studs are placed the 3″×4″ bearing pieces, which are prevented from turning over by means of the $\frac{7}{8}$″×4″ braces *c* secured with nails. As it is important to bring the forms to a true level, a double adjustment wedge is provided at the bottom of the studs. This wedge, upon being driven up or slackened, will give the necessary adjustment.

The forms for the columns are made of 1½- or 2-inch material, it being best to use the thicker plank for this construction. The planks *d* for the column forms are run vertically, and the several planks making up the sides of the forms are battened together, as shown at *e*. The planks are nailed along the edges, and are prevented from spreading by means of braces, or yokes, *f* placed at intervals of 2 or 3 feet. These yokes are constructed by simply crossing the pieces at the corners, nailing them securely, and clinching the nails. The tops of the column forms are cut out so as to allow the beam and girder forms to enter and bear upon the upright planking. These openings in the columns are additionally reinforced by means of the pieces *g*.

17. In the construction of the beam and girder forms, 2-inch planks *h* are generally used for the sides. In order to form a chamfer on the lower edges of the beams and girders, triangular fillet pieces *i* are nailed in the forms. The edges of the beams and girders may be stop-chamfered by cutting off the fillet pieces near the intersections and columns.

Owing to the depth of the beams and girders, the sides of the forms must usually be made of two planks. The different depth of the beams and girders can be provided for by blocking up the bottom piece *j* at an even distance from the horizontal cap piece at the top of the supporting stud. In order to prevent the side forms of the beams and girders from spreading, cleats are nailed on the horizontal cap piece, and wedges *k* are driven between this and the form boards.

It is customary in this form of construction to make the forms for the slabs of $\frac{7}{8}$-inch plain boards, frequently using tongued-and-grooved material. These slab forms are held together by battens *l*, and are supported along the edge of the beam and girder forms by means of the 1-inch cleat, or strip, *m*, and intermediately by 3″×4″ or 3″×6″ joists *n* that rest on the rabbeted batten secured to the forms.

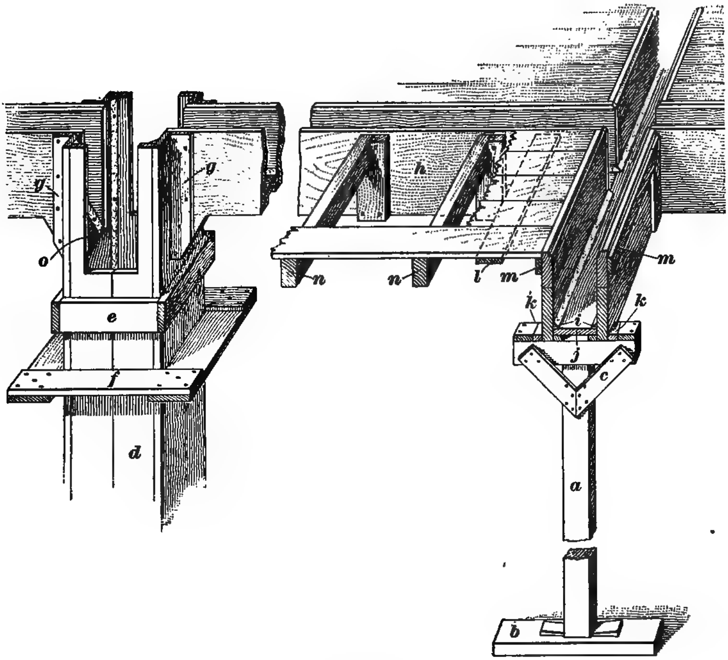


FIG. 6

18. This style of form has been used successfully, and produces work of good appearance. It is also fairly economical, as the slab forms may be used for the several panels in the different stories of the building. If the distance between the beams is considerable, the slab forms should be constructed in several parts so that they may be readily handled in taking down and resetting. By making the side forms of the beams of a depth that will accommodate the largest size beam or girder on the job, the same forms may be used for any of the smaller beams by blocking the bottom boards at different heights, as previously explained.

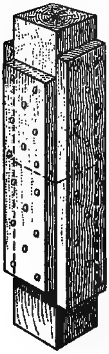


FIG. 7

As will be observed on referring to Fig. 6, the pieces *o* at the junction of the girders and the column are set at an angle with the bottom pieces *j* and the column planks *d*. This construction will produce a joint similar in appearance to that of a corbel.

19. If, in building up forms, the upright studs are not long enough, they are sometimes spliced, as shown in Fig. 7. Although such practice is probably necessary in some cases to secure economy, it is not recommended, because it is difficult to get a good square bearing where the two pieces of the studding abut. The weight of wet concrete is considerable, and if the ends of the studs bear only on an edge, there is a tendency for them to buckle, or give, and unless the splice pieces on the sides are long, and securely nailed, such supports are an element of weakness and are liable to cause a serious collapse. If such splices are made in the supports, it is well to brace them against possible failure with ledger boards or braces.

20. Forms for Tile and Concrete Floor Construction.—In Fig. 8 is shown a type of form that has been successfully used in conjunction with the composite system of construction known as the reinforced hollow-tile floor construction and described in connection with the Kahn and Gabriel systems in a preceding Section. The side and bottom forms of the main girders are built of material dressed to 1⅜ inch. These forms are supported upon 3″×4″ posts carrying cap pieces *a*. The girder forms are held from spreading at the bottom by cleats spiked to the cap pieces, between which the side forms are clamped by means of wedges *b*.

Instead of the usual battened plank or flooring construction necessary for the formation of a concrete slab, 2″×8″ planks *c* are used. These planks, instead of being placed close together, are laid 8 inches from edge to edge, the tile spanning the open space and the plank forming the bottom of the form for the concrete that is to be placed between the tile. These planks are supported at the end by a ledger board *d* nailed to the side of the girder forms, and are held intermediately by a bent. This consists of 3″×4″ uprights, braced intermediately with a 2″×8″ piece. Across the tops of the uprights is a 2″×8″ string *e* that supports 2″×8″ joists, on which rest the flat planks that support the tile and concrete construction.

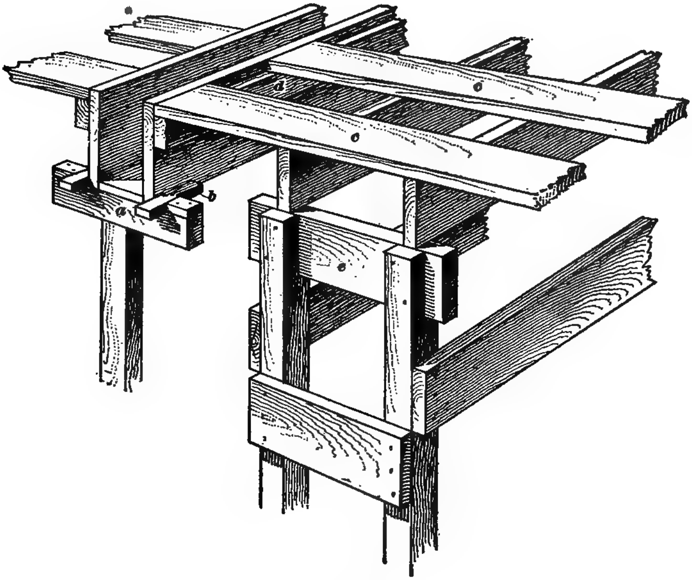


Fig. 8

In this type of form construction, the material is economically used, and, besides, it is of nearly one size. As will be observed, 2″×8″ pieces predominate, and these, together with the 3″×4″ studs and the 1⅝-inch stock used for the girder forms, make up most of the bill of material required.

21. Forms Constructed of Light Material.—The system of form construction illustrated in Fig. 9 was successfully used in the erection of several large reinforced-concrete buildings. Referring to view (*a*), the sides of the girder forms are constructed of ⅞-inch dressed tongued-and-grooved flooring boards, and the bottom of the forms consists of a 2-inch plank. The sides of the forms are secured with batten boards *a* and *c*. The top batten acts as a support for the 2″×6″ joists *f* that support the slab form, and some of the weight of these joists is transmitted by the upright pieces *b* to the lower batten board *c*. The slab form consists of ⅞-inch material laid upon the top of a 2″×6″ cross-joist. The plank or board nearest the beam form is secured to the top of the sides of the forms.

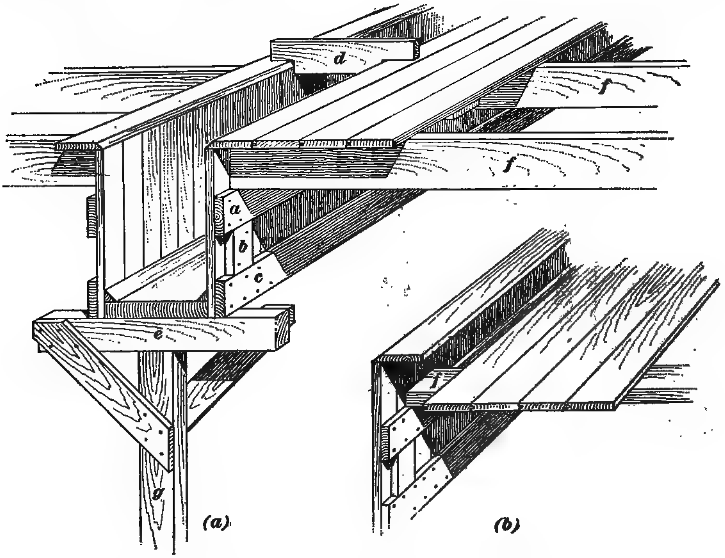


FIG. 9

In order to prevent the warping or twisting of the side forms of the beams or girders, and to hold them in the proper positions, separating pieces *d* are provided at intervals. These pieces are removed as the concrete approaches the top of the forms. The sides of the forms at the bottom are secured to the bottom plank of the forms with screws or nails, and the entire centering is supported by means of 3″×4″ studs *g* and a cap plank *e*, the studding being placed at intervals of about 5 feet.

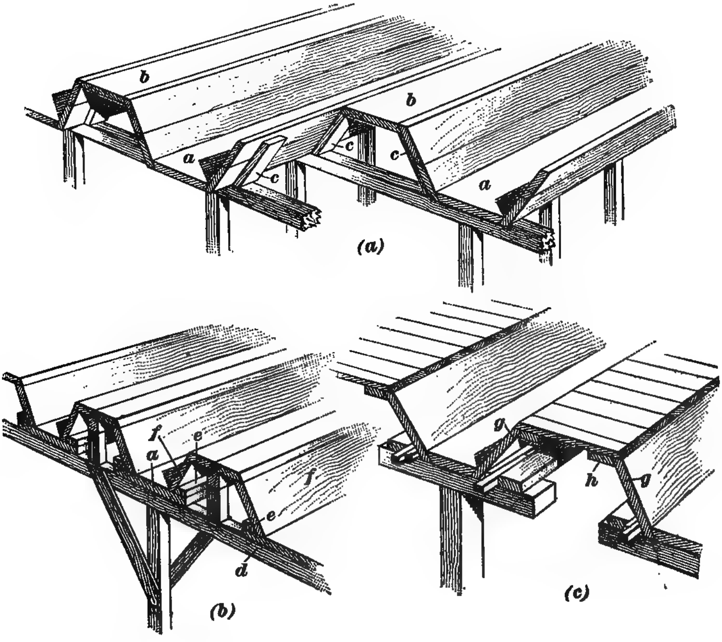


Fig. 10

In demolishing or taking down the form work, the 2″×6″ joists *f* are turned flat upon the ledger board *a*, so that the slab form is lowered as indicated in (*b*).

This type of form work has the advantage of being mostly constructed of light material; besides, as the sides of the forms of the beams and girders are constructed of matched flooring, the work will be smooth and true to line. There is also an advantage in the way in which the slab form may be dropped and thus allow the concrete to dry out without removing the forms entirely.

22. Forms for Trough Construction.—In erecting the form work for the trough system of reinforced-concrete construction, the forms are arranged so that they will be interchangeable, as illustrated in Fig. 10.

The best method of arranging the construction is illustrated in view (*a*). Here, the lower portion *a* of the form is of the same section as the top section *b*; thus, the form work is completed when the form *b* is superimposed on the form *a*, and is held in place by the battens *c c*.

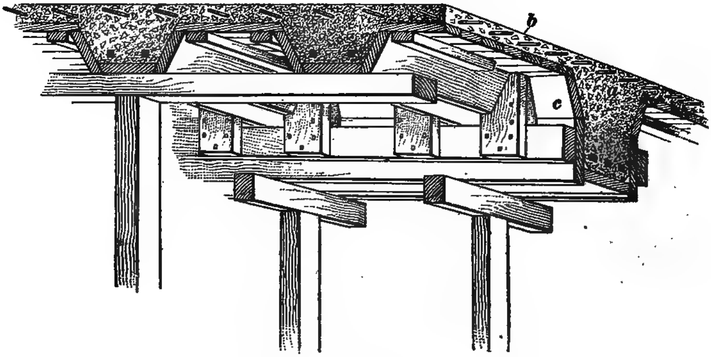


FIG. 11

Another way to form the center for this type of construction is shown in view (*b*). Here, the bottom portion of the form consists of a 2-inch plank *a* supported upon 4″×4″ studding by a cap piece, or stringer stud, *d*. The planking is provided with a continuous strip *e* along the edges, so as to hold the inverted trough *f* rigidly in place.

Frequently, it is desired to space the beams farther apart than shown at (*a*) or (*b*), in which case the forms are constructed as shown in (*c*). The sides of the beam forms are shown at *g*, while the slab connecting the concrete-beam sections is formed upon form work made of $\frac{7}{8}$-inch dressed tongued-and-grooved material nailed together so as to cover the distance between the trough forms. Ordinarily, this distance is not sufficiently great to require any support between the beam forms, as the flooring and battens *h* will carry the load of the wet concrete between the points of support.

23. In Fig. 11 is shown a section and perspective view of the form work of a typical trough construction. The widening of the upper part of the girder is arranged in the manner shown at *c*, so as to provide the necessary concrete in compression in the upper portion *a* of the girder section. This is necessary because the slab *b* is made so thin in order to attain economy that it will not provide the necessary compressive resistance at the top portion of the girder.

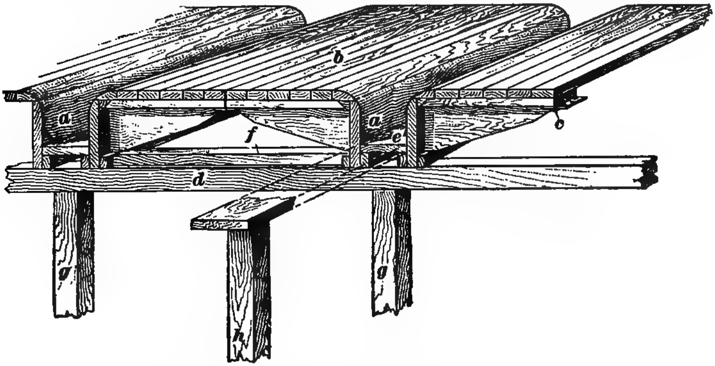


FIG. 12

By arranging the form construction in the manner shown, it can be readily taken down and put up with a minimum amount of labor, the use of nails being greatly reduced.

24. Collapsible Forms.—Several attempts have been made to arrange the forms for a reinforced-concrete floor system so that they will partly collapse and thus be easy to take down or away from the concrete as soon as it has properly set. One type of collapsible form is shown in Fig. 12. In this construction, the side forms for the beams are embodied with the slab centering by the construction of a collapsible box, which furnishes a form that is the exact shape of the space between two adjacent beams. The vertical sides of the box *a* make the side forms for the beams, and the horizontal portion *b* of the forms supplies the centering for the slab. These boxes are arranged with a hinge *c*, and when in place they rest upon cap pieces *d* supported by studs *g*. The form is prevented from collapsing by the bottom plank *e* of the beam forms, against which it bears, and by the strut *f* that is wedged between the sides of the form.

In order to take down the forms, the wedges at the bottom of the upright supports *g* are removed and those at the ends of the pieces *f* are driven out. The struts are then removed, and the form is allowed to double up on the hinges *c* and thus release itself from the concrete. After the collapsible centering has been removed, the planks beneath the soffit of the beams and girders may be retained in position by means of struts *h*.

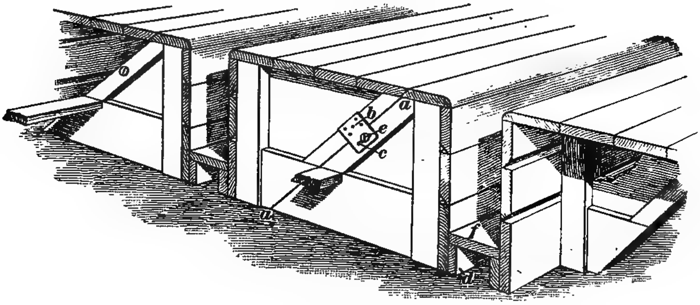


FIG. 13

25. Another type of collapsible form is shown in Fig. 13. In this form, the box that supplies the form work for the sides of the beams and the centering of the slabs, instead of being hinged, is cut through along the line *a a* and is held in position by the slotted iron plate *b* and screw *e*. By releasing and loosening the screw, it will slip in the slot *c*. Then one-half of the form can be drawn downwards, so that it will recede from the face of the beam and thus allow the form to free itself from the concrete and be taken down. It will be observed that the sides of the collapsible forms make the sides for the beam forms, the battens *d* furnishing the support for the bottom form board *f* of the beams.

Although there are some advantages in the use of collapsible forms, it is doubtful whether they are always as economical in operation as are the ordinary types of form work.

26. Heavy Construction—Partly Collapsible Forms.—In Fig. 14 are shown the details of a form construction for a reinforced-concrete floor system that is particularly well constructed and at the same time capable of being used economically. It is built on the collapsible principle, so that it may be readily set up and taken apart.

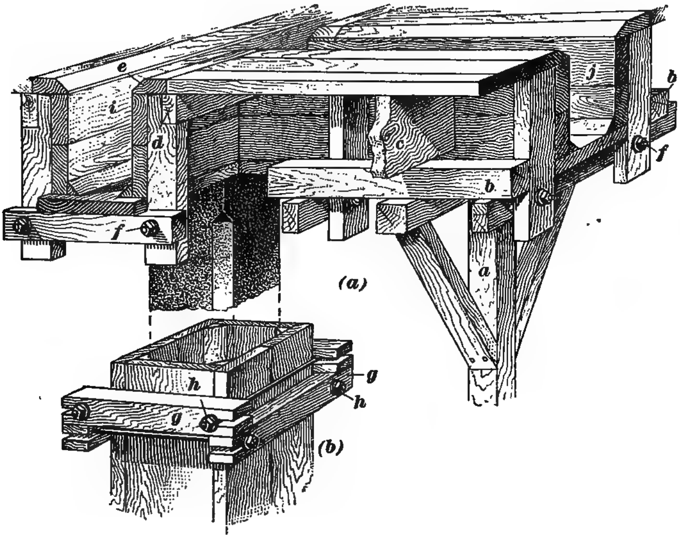


Fig. 14

In this form work, the sheathing of the beam and girder forms and the plank used for the slab centering are 2 inches in thickness. The forms are supported by 4″×4″ studs *a*, view (*a*), placed at intervals of about 3 feet. These studs are provided with a 4″×4″ crosspiece, well braced with 2″×4″ pieces, and project sufficiently beyond the sides of the beam forms to carry loose ledger pieces *b*. These ledgers support the joist *c*, which holds up the plank of the slab centering.

It will be noticed that the construction of the form around the beam, as shown at *i*, is somewhat different from that around the girder, as shown at *j*. Where the joists run at right angles to the sides of the girder forms, as at *j*, they provide necessary support for the slab centering close up to the sides of the forms, but if the joists run parallel with the form, as the form for the beam *i*, then a 2″×4″ piece *d* must be provided to support the ends of the slab centering and the beveled strip *e*. In order to remove this centering, the 4″×4″ ledger piece and the joists are taken away, so as to permit the slab centering to drop. Then the forms may readily be taken down by opening them away from the sides of the beams or girders, the bolts *f* acting as hinges around which the halves of the form swing on the lower ends of the battens.

In the construction of the column forms, the planks are 2 or 3 inches in thickness, and are placed vertically, as shown in Fig. 14 (*b*). The planks on each side of the columns are battened together with 2″×4″ battens securely nailed, and the sides of the forms are prevented from spreading by the 4″×4″ yokes *g* and the tie-bolts *h*. These yokes, or binders, are formed by boring and slotting the ends of the 4″×4″ pieces, so as to receive the ½-inch tie-bolts *h*, which are threaded and provided with malleable cast washers and nuts.

27. It will be noticed that the forms are made so that the corners of the columns, beams, and girders are cast with a chamfer; also, there is a fillet in the corner made by the junction of the concrete slab with the sides of the beams and girders.

The form construction just described has proved very satisfactory, and is economical on account of the facility with which it may be removed and readjusted. Although the use of 2-inch plank for the slab centering is unusual, it is doubtful whether the expense is materially increased over the use of 1-inch material, because the 2-inch plank is more likely to remain intact with hard usage and is worth more to sell when the work is completed and the centering material is to be disposed of. By using the heavier plank for the slab centering, much better work is likely to result, as there is less sagging and warping and the boards are not splintered nor so badly broken as when lighter material is used.

28. Arrangement of Forms for Stopping Work. As it is frequently impossible to fill the forms with concrete in a single operation or a single day, it is necessary to arrange stops, or dams, in the form work, so that the concrete of the successive operations will make proper and structural junctions. There is considerable difference of opinion as to whether the concrete of a floor system should be stopped at the center of the span of the beams or along the central axes of girders and beams and at the bearings of beams with girders and columns. Generally, the concrete is stopped off according to the latter method, by using wooden partitions arranged as shown in the plan view, Fig. 15. In this figure, the beam forms are shown at *a*, the forms for the girders, at *b*, and the position of the column forms, at *c* and *d*. As a rule, the concrete for the slab is stopped at the center line *e e* of the girder, and built-up wooden stops *f* are arranged to provide pockets in the concrete of the girder so that the beams *g*, when subsequently filled with concrete, will have a bearing upon the girder.

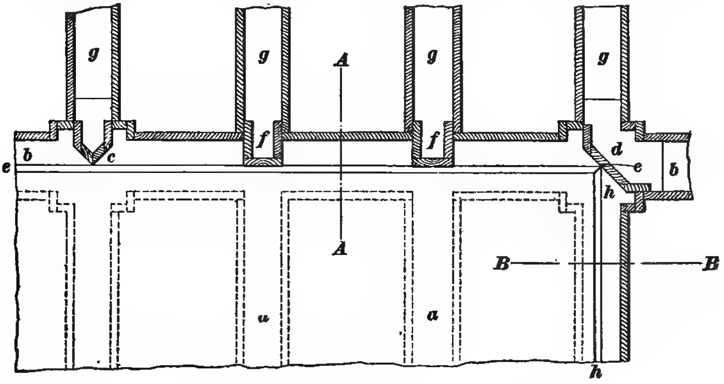


FIG. 15

If, in stopping off the work, it is merely necessary to provide a beam bearing on the column, the construction shown at *c* is generally followed. Here, the stop is constructed so as to cut into the solid concrete of the column filling as little as possible and still provide an adequate bearing for the beam *g*. If it is necessary to stop both the beam and the girder over the column, forms are arranged as shown at *d*. In all cases, the girder is filled at least to the bottom of the slab, the double lines *e e* and *h h* representing the beveled edge of the slab where it is stopped off. This is more clearly illustrated in the sections shown in Fig. 16. View (*a*) shows the section through the girder, and view (*b*), a section through the beam. These sections are taken on the lines *A A* and *B B*, respectively, Fig. 15. Sometimes the concrete is stopped off at the center of the span of beams or girders, and frequently a concrete block that has been previously molded is used to act as a dam. This block is arranged to fit around the steel reinforcement and is incorporated with the concrete work when it has set.

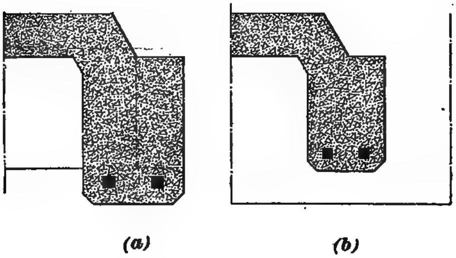

(*a*) (*b*)

FIG. 16

Sheet-metal partitions may also be used to stop off the concrete work. Wood, however, is usually more available.

FORMS FOR CONCRETE WALL CONSTRUCTION

29. Methods of Construction.—In the construction of reinforced-concrete walls, it is necessary to use forms that will permit the walls to be built rapidly, plumb, and of equal thickness throughout. There are two distinct methods of form construction for concrete walls. In one, the forms are built to form the complete wall for one floor in one operation, and in the other the forms are arranged so that the wall is carried up in courses, several feet at a time.

In this latter construction, the forms are taken away as the concrete obtains its initial set and replaced upon the top of the wall, so that fresh concrete can be placed. The advantage of this method over the former is that much less lumber is required. Besides, the forms do not have to be so strongly braced, because, as the walls are carried up only a few feet, the hydrostatic pressure of the concrete on the sides of the form is not so great as it would be if 10 or 12 feet of wall were constructed at one time.

30. Wall Forms With Wire Ties.—A type of wall-form construction that is frequently used is illustrated in Fig. 17. It consists primarily of 3″×5″ or heavier uprights *a* and planks *b*. The uprights are spaced from 2 to 3 feet apart and act as battens, or braces, for the planks *b* of the wall form. In order to prevent the sides of the forms from spreading when the concrete is tamped in place, a wire tie *c* is used. This tie is looped over the one upright, then brought through the forms and twisted around the upright on the opposite side of the wall, and finally made taut by twisting with a bar or stick *d*. To keep the form boards the proper distance apart for the thickness of the wall, a block, or stick, *e* of wood cut to a length equal to the thickness of the wall is sometimes inserted. This separator may be knocked out of the wall before the concrete has finally set and after the forms have been raised, or it may be left embedded in the concrete. Owing to the difficulty encountered in removing a wooden separator without damaging the face of the work, a wooden piece *f* is frequently used at the top of the forms instead. The concrete of the portion of the wall that has obtained its

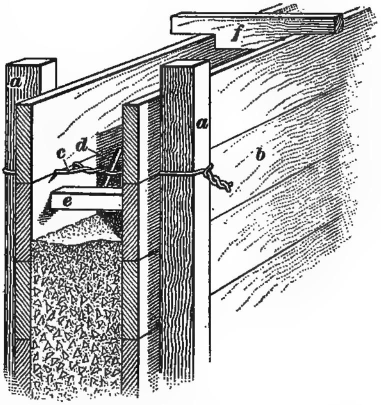


Fig. 17

initial set acts then as a separator at the bottom of the forms; this cleat is readily removed before the forms are raised to the next course, the concrete of the course above filling the space left by the separator in the course below.

31. Wall-Form Construction With Clamp Bolts. A wall-form construction similar to the one just described is illustrated in Fig. 18. In this form, however, a clamp bolt *a*, instead of a wire tie, is used to prevent spreading. This bolt is formed on one end with a square hook *f* that fits over the upright support at one side of the form. It passes through the form, and together with a washer plate *b*, a block of wood *c*, and the threaded wrench nut *d*, forms an adjustable clamping device. By turning the wrench nut, the form boards can be brought to any required distance apart and secured tight against the wooden separator *e*. If a bolt of this character is used, it must be knocked out before the concrete has finally set and when the form boards are to be raised to form the next course of concrete. The bolt is preferable to the wire tie, because it is removed from the concrete. Wire ties are usually cut off close to the concrete work after the form boards have been removed, and as the ends frequently project, they rust and thus stain the wall.

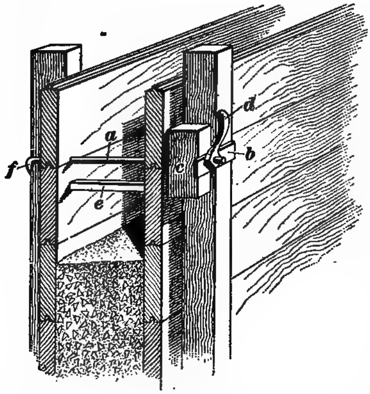


Fig. 18

If tie-bolts are permitted to remain in the concrete too long, considerable difficulty will be experienced in removing them. Where the walls have a thickness of more than 6 inches, it is frequently difficult to remove the bolt after the concrete has set. In some cases, the bolts are turned at intervals while the concrete is setting, in order that their bond in the concrete will be destroyed and their removal accomplished more readily. If some precaution is not taken, it is frequently necessary to use a heavy hammer to start and drive the bolts from the concrete. This operation requires considerable time, and besides is liable to damage not only the thread on the bolt, but the wet concrete as well.

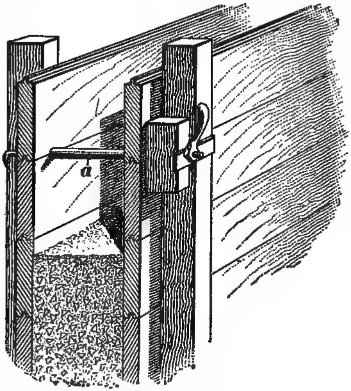


FIG. 19

32. Wall-Form Construction With Pipe Separators. — In conjunction with the clamp bolt just described, a pipe separator is sometimes used to hold the form boards at the proper distance apart. In Fig. 19 is shown the construction of a wall form in which a pipe separator *a* is used with the clamp bolt. The pipes may be driven out of the concrete after it has obtained its initial set, or they may be left in place. In either case, the hole is filled with cement. It is best, however, not to leave the pipe in the concrete because its ends will probably rust and thus stain the wall.

33. Wall-Form Construction With Concrete Separators.—One of the best methods of constructing wall forms is to use concrete separators. These separators are usually square or cylindrical in form, with a bolt hole through the center, and are molded before the wall is built. The separator shown in Fig. 20, known as the **McCarty separator,** may be used in constructing the form work illustrated in Fig. 21. In this figure, as in the preceding examples, the wall planking is held in place by uprights *a*, which are arranged to receive a tie-bolt *b*. This tie-bolt is slipped through the form boards and concrete separators *c* that have been previously cast.

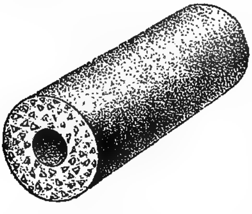


FIG. 20

These separators are to be had in 4- and 6-inch lengths. Therefore, many thicknesses of wall can be built up by combining the lengths. For example, in an 8-inch wall, two 4-inch separators may be used; in a 10-inch wall, a 6-inch and a 4-inch separator; in a 12-inch wall, two 6-inch separators; and in a 14-inch wall, two 4- and one 6-inch separator.

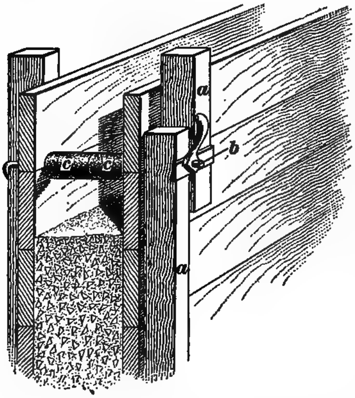


FIG. 21

34. Cast concrete blocks are also used for leaving holes in concrete floors or walls, in which case they are used in the manner shown in Fig. 22. In this figure, the form board of a concrete slab is shown at *a*. The cast concrete block, previously molded, is secured to the form by means of a spike or nail, which is driven through the center, a small sheet of tin or piece of wood *b* serving as a washer. A hole of any moderate size may be cast in this tapered concrete block, or, if a larger hole is desired, the entire block may be knocked out after the concrete surrounding it has set, as it is sufficiently tapered to allow this to be done. In any event, the material in the block, being of sand and cement, can be more readily cut than can the reinforced stone concrete surrounding it. The advantage of using concrete separators and sleeves is that they do not cause spots to form on the wall or ceiling.

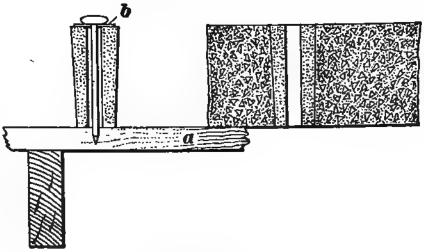


FIG. 22

35. Steel-Clad Self-Alining Forms. In Fig. 23 is shown a patented form construction, known as the **Universal steel-clad self-alining form.** This form work is suitable for wall and partition construction. It consists of typical units *a* that are well strengthened and constructed. These units are tied together at the corners with iron clips, and each is covered with sheet steel on the side against which the concrete work is placed. The metal sector, or clip, *b* at each corner is a particular feature of the unit. This sector is arranged so as to present a conical surface when combined at a corner with the three sectors of adjacent units; or, when two of them, together with a filling piece, are joined, as shown at *c*. Over this conical projection a dished washer, as shown at *f*, Fig. 23 (*b*), is arranged to fit, and the several units of the wall form are drawn and tied together and prevented from spreading by the bolts *d*. These bolts pass through a piece of pipe *h*, 1 inch shorter than the thickness of the wall, the washers on the inside of the forms making up the difference.

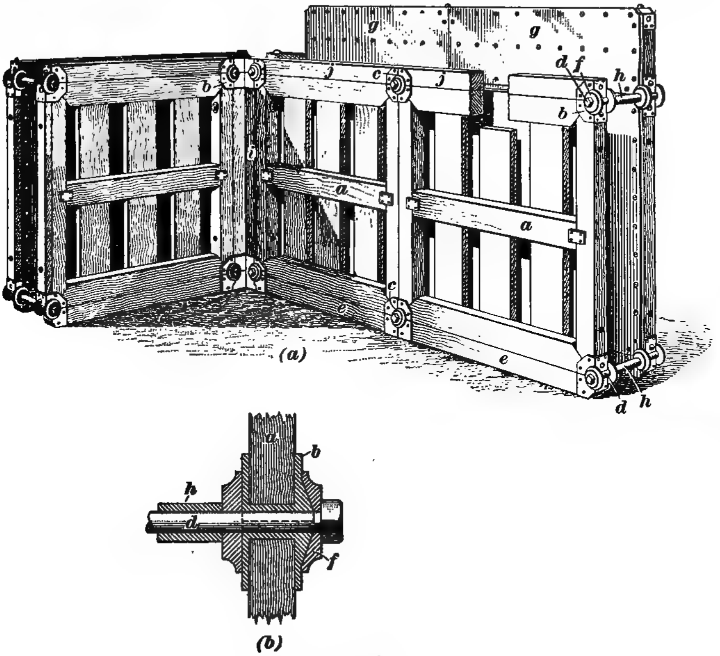


FIG. 23

As arranged in Fig. 23 (*a*), the forms are one unit in height and can be placed so as to turn a right angle. At *e* is illustrated what is known as a *3-inch unit*, which is employed as a basis for starting the form construction. Filling units *j* are employed to increase the height, so that additional concrete may be placed for tamping. *One-foot units g* are shown on the outside form. These are used to prevent the concrete from being thrown over the top of the form when it is dumped from barrows. At *i* is shown what is known as an *inside corner piece*, and a similar piece is provided for the outside corner construction.

36. The advantages claimed for these wall-form units are as follows: Their first cost is low; they are practically indestructible and light in weight; they can be set up rapidly and are stiff and rigid when in place, because each unit is of considerable area; they are interchangeable and can readily be transported and stored; the forms made up of them are lined with steel plate and consequently give a very smooth finish to the concrete work; they are substantially framed in such a manner as to use narrow strips with little shrinkage across grain; and by the use of the square and octagonal adjustable corners for both inside and outside angles, and because of the filler pieces, the form work for any desirable shape of wall can be readily set up.

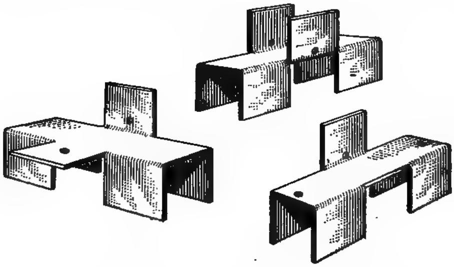


FIG. 24

37. Clamping Devices and Plank Holders for Wall Forms.—Many devices that aid in the construction of reinforced-concrete walls have been invented. One of the most useful of these devices is the **Sullivan pressed-steel plank holder,** various forms of which are shown in Fig. 24. As will be observed, these holders are formed from an iron plate by shearing and bending it so as to form clips to hold the upper and lower planks of a wall form in their true positions.

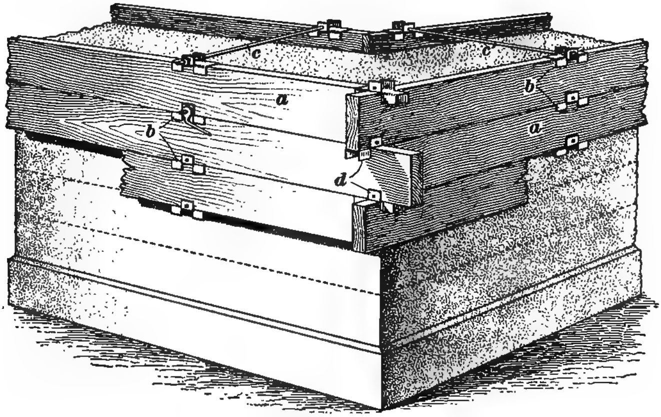


Fig. 25

The application of this type of plank holder is illustrated in Fig. 25, in which the wall-form boards are shown at *a* and the plank holders at *b*. The two lower clips of the plank holder fit over the lower form board, while the middle clip holds the

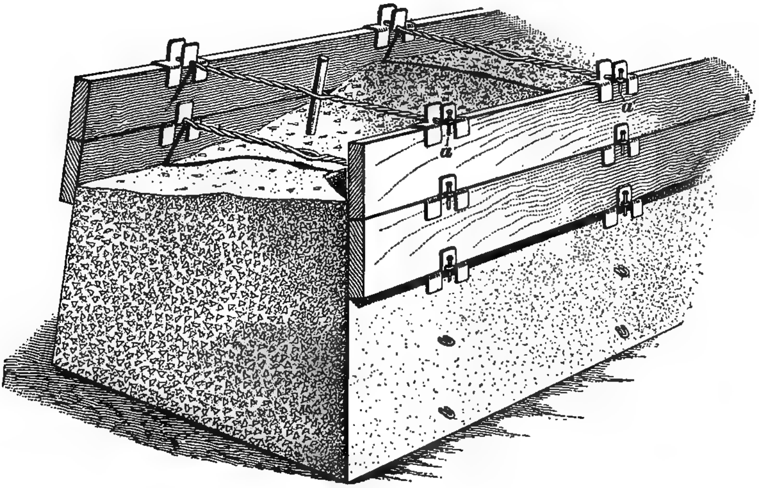


Fig. 26

next higher form board in position and also acts as a washer for the tie-bolt *c*. The plank holder, as shown at *d*, can also be used to hold intersecting corner boards in position.

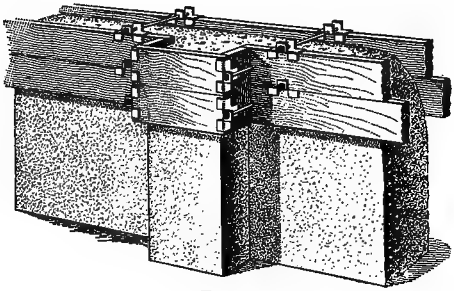


Fig. 27

38. If it is not desirable to use tie-bolts with the clips, or plank holders, wire ties may be used, as shown in Fig. 26. The loops of the wire ties are passed through the holes left in the clips and secured with nails, as shown at *a*.

The convenience with which these plank holders may be used for the construction of almost any wall is well illustrated in Fig. 27, which shows the wall forms in position for the construction of a concrete wall with a pilaster. As shown, the clips are conveniently used to bind the corners of the forms for the casting of the pilasters, as well as to hold the form boards in the straight portion of the wall.

The Sullivan steel plank holder may also be used for the construction of column forms, as shown in Fig. 28. In this case they rest on the corners of the form boards and provide lugs for the tie-bolts used to bind the form together.

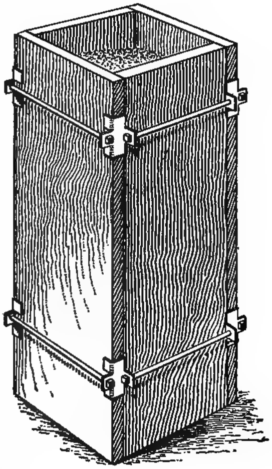


Fig. 28

39. Another patented device, known as the **Dietrich clamp,** has merit in wall-form construction. This clamping device is illustrated in Fig. 29, and consists essentially of three pieces, namely, the side plate, or clamp, *a*; the hook plate *b*, which is pivoted to the side plate by means of the rivet *c*; and the steel wedge *d*, which is arranged to fit the slots *e* in the hook plate.

The use of this device is clearly illustrated in Fig. 30. In (*a*) is shown a partly constructed concrete wall with the form boards in place. These boards are slotted near the edges, and through these slots the clamping device is inserted from the inside, when it occupies the position shown at *a*, in views (*a*) and (*b*). The wire tie with the looped ends is caught by the clamping device, as shown in (*b*), and then the wedges *b* are driven in place.

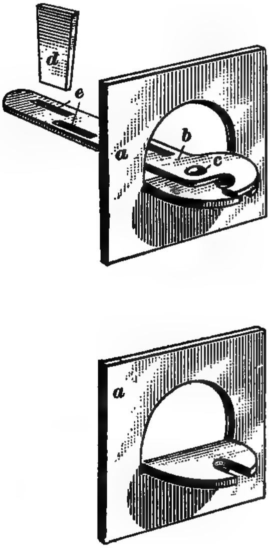


FIG. 29

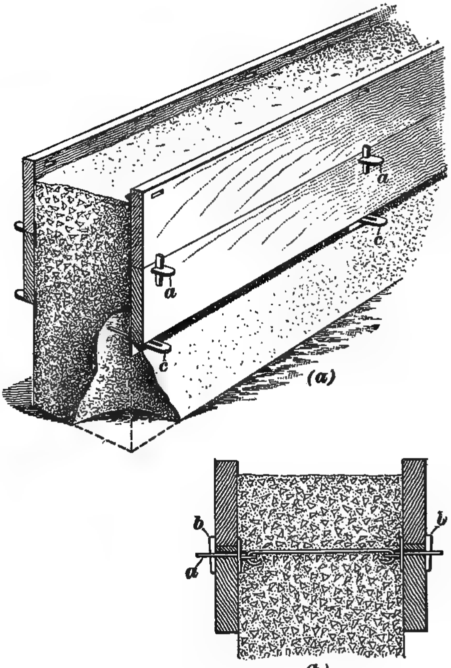


FIG. 30

One advantage of this device is that when the wedges are driven out, the form boards may be removed and the clamp disengaged from the wire or rod tie by swinging it around the rivet *c*, Fig. 29, thus leaving this tie embedded in the wall. The small holes left by the clamping device may then be grouted. Besides serving to clamp the form boards in position, the Dietrich device may also be used to support them, as shown at *c*, in Fig. 30 (*a*).

40. Wall-Form Construction With Battened Panels.—In the several types of wall-form construction already described, single planks, set one upon the top of the other, are used. Although this method of erecting form work serves very well for small work, such as house construction, it is desirable with larger and higher walls, such as would exist in manufacturing or commercial buildings, to carry on the work more expeditiously. To accomplish this, the walls are carried up in stages of 5 or 6 feet by using panels made up of three or more planks. These are battened together and are held in position with wood or iron guides and tie-bolts. This type of wall-form construction is illustrated in Fig. 31, a side elevation of the construction being shown in (*a*), a plan in (*b*), and section through the form construction in (*c*). The several panels *a* are constructed of 1½-inch plank that is well battened together. These panels are held between guides *b* constructed of two 2″×6″ pieces. At the lower end of the guides are placed 1½-inch plank separating blocks, and at the top a 1½″×6″ plank tie *c* is well nailed to the uprights. In this way, the tie-bolts *d* for the forms may be placed in any convenient position throughout the length of the guide pieces.

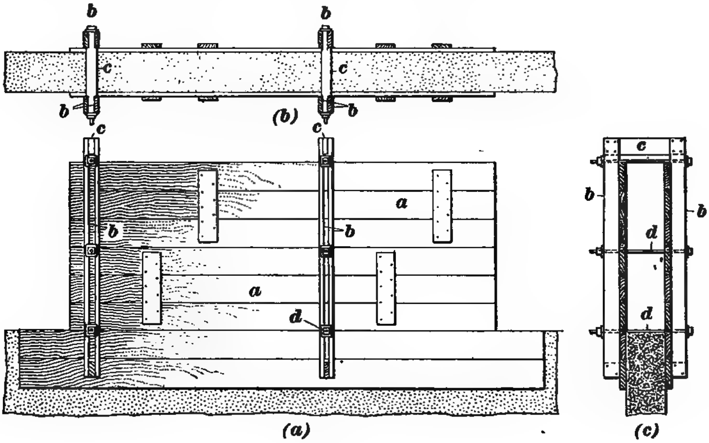


FIG. 31

41. In operating this type of form, the side panels are put in place and the tie-bolts *d*, Fig. 31 (*c*), are tightened up. After the concrete has set, the bolts are withdrawn and the form is raised to the position shown, the bottom bolt resting upon the top of the finished concrete and a considerable portion of the form extending down over the finished wall. Thus, the form is not only supported, but it is rigidly held in position plumb over the finished portion of the wall. The form is again filled and the operation repeated.

Tie-bolts used in this manner, or as described in the preceding articles, should be well soaped or greased, as this facilitates their withdrawal from the concrete.

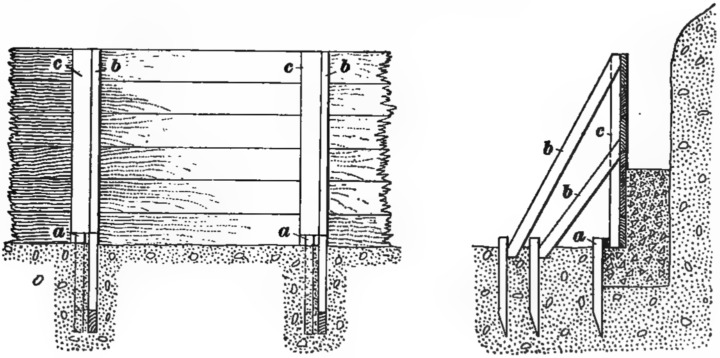
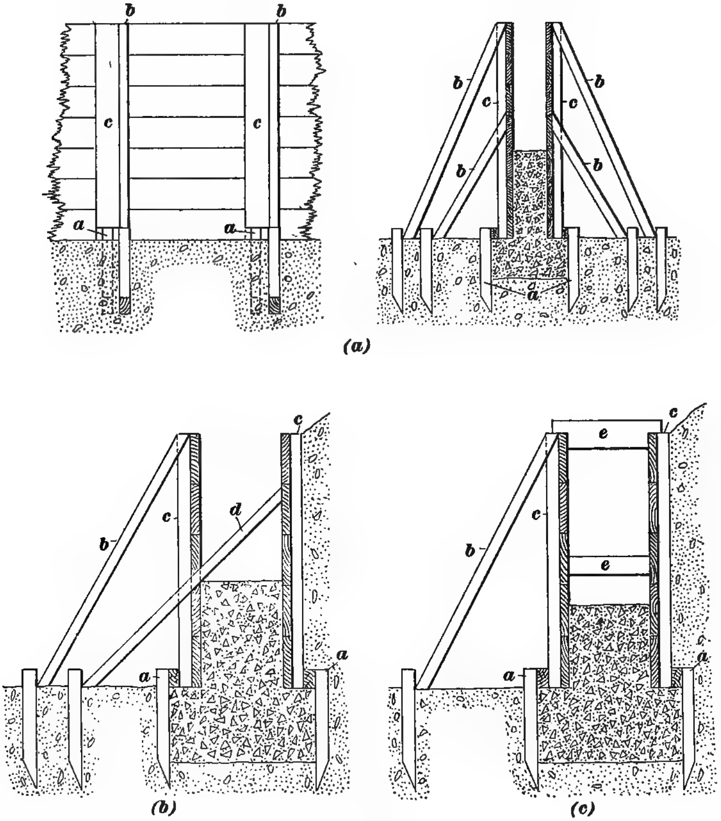


FIG. 32

Instead of the wooden guides for the forms, steel channels placed back to back are sometimes used. These are held sufficiently far apart for the passage of bolts by packing or filler pieces that are riveted or bolted in place. Such guides are very serviceable, and as they are much stiffer than the wooden guides, they prevent the wall from bulging. For 1½-inch plank, these guides, or braces, should not be placed farther apart than 4 feet, and if ⅞-inch material is used for the construction of the panels, they should be placed about half this distance apart.

42. Form Work for Area and Retaining Walls.—In the construction of area and retaining walls, the form work usually consists of either light or heavy sheathing that is well braced with back braces, as shown in Fig. 32. In this figure, an earth embankment acts as one side of the form, and heavy planking extending horizontally is used for the other side. This planking is spiked against heavy uprights *c*, which are prevented from moving at the bottom by blocking placed between sheet piling or heavy stakes, as at *a*. The form is well back-braced, as shown by the braces *b*, which are well secured at the foot by means of heavy stakes. The braces may be held by sheet piling that is properly driven or by sill



(a)

(b)

(c)

Fig. 33

pieces set in the earth. Sometimes they are made adjustable by wedging.

43. If a wall is to be constructed in a place where there is no embankment, a double set of forms must be used, as shown in Fig. 33 (*a*), in which similar letters to those in Fig. 32 (*a*) represent similar parts. If the wall is of considerable height and is filled at one time, the hydrostatic pressure of the concrete will be considerable. In such cases, heavy planks that are well tied and strongly braced must be used for form work.

If the soil of an embankment is unstable, it will be necessary to sheath and brace it and at the same time arrange the forms for the concrete work. Where this condition exists, either the back, or oblique, braces *d* of the forms, Fig. 33 (*b*), must be extended through the forms and cut off after the concrete is placed, or the back, or oblique, braces *b*, Fig. 33 (*c*), must be placed on the side form away from the wall and the sheathing against the embankment braced to the outside form with horizontal pieces, or trench braces, *e*. These horizontal pieces are removed as the concrete is placed and reaches their respective levels.

All form work of this character requires the use of at least 2-inch—preferably 3-inch—planking, as well as braces of heavy timber, such as 6 in. × 8 in., 8 in. × 10 in., 10 in. × 12 in., and heavier, as the conditions warrant.

44. Forms for Hollow-Wall Construction.—The casting of hollow reinforced-concrete walls is extremely difficult. Two methods are commonly employed in the construction of forms for such walls. One method consists in using a rigidly constructed core box for casting a hollow space in the wall. This box is of such shape that it can be drawn up with the outside form. The other method consists in using a collapsible core box; that is, a box that can be made smaller by removing wedges or other devices, so that its position can be readily changed or it can be easily withdrawn from the concrete.

45. The first method of constructing a hollow concrete wall is illustrated in Fig. 34, in which a plan of the construction is shown at (*a*), an elevation of the side forms at (*b*), and a section through a partly constructed wall and the forms at (*c*). From these views, it will be observed that the outside forms are constructed with double guide, or brace, pieces that are connected at the top with tie-pieces and fastened at the middle with through bolts. The core box *a* is constructed of light material that runs vertically and is nailed to battens, or braces. It is arranged so as to cast hollow spaces in the wall, leaving through divisions of concrete connecting the two sides of the wall, as shown in (*a*). This core box is made tapered or given a draft so that it can be readily drawn from partly set concrete. The core box is held in position by means of rods that pass through clips or staples at the top. These rods rest upon the top edges of the outside forms, and thus suspend the core boxes.

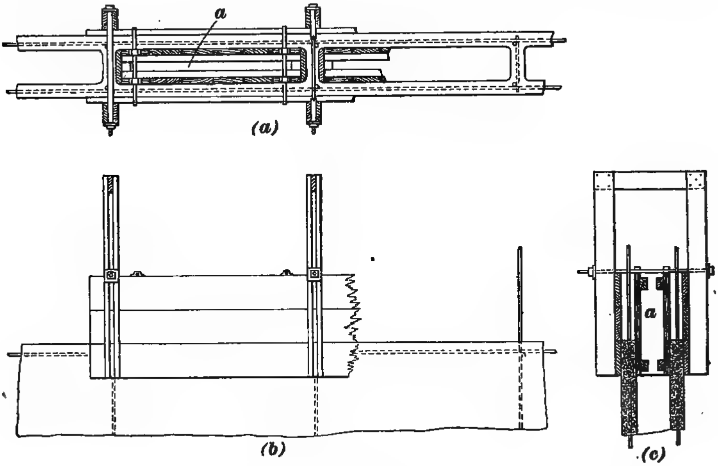


Fig. 34

As will be observed from view (*a*), the outside guides for the forms are placed opposite the partition of concrete that connects the two sides of the wall. The forms are arranged in this manner so that there will be no tendency to crush the partly set concrete wall when the tie-bolts are tightened.

46. The second method of constructing a hollow concrete wall is shown in Fig. 35, which illustrates a collapsible core box. The box is arranged with end and side pieces, the side pieces being kept at the proper distance apart by the cleats *a*, which are pivoted on bolts, as at *b*. The ends of these cleats fit between the end forms *c* and the battens *d* that hold the planks of the sides of the forms together. The core box is collapsed by turning the cleats *a* on the bolts so that they are free of the slot. This allows the sides of the forms to come together slightly, thus reducing the width of the core box and allowing it to be readily withdrawn from the concrete.

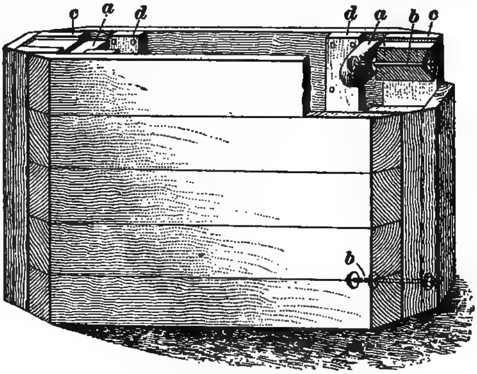


FIG. 35

Numerous other devices have been arranged for making core boxes collapsible. These depend on the loosening of bolts or screws or on the use of wedges and keys that can be backed up by a blow from a hammer. The time spent on the careful design of collapsible core boxes for concrete wall construction is amply repaid by the time gained in the constructing of the wall.

47. Metallic Forms for Hollow-Wall Construction. A type of sheet-metal form construction known as the **monolith concrete form** is illustrated in Figs. 36 and 37. The method of casting the concrete form with cylindrical hollow spaces extending vertically is clearly shown in Fig. 36, while the details of this form construction are shown in Fig. 37, which is a cross-sectional view through the wall form.

The sides of the forms consist of No. 16 gauge galvanized sheet iron, riveted to 1″×1″ angle iron *a*. These forms are made in unit sizes of 12 in. × 24 in. and 24 in. × 24 in., so that they will be convenient to handle. The cores *b* to form the hollow spaces in the wall are made of either metal or wood—generally the latter. In casting walls greater than 10 inches in thickness, cylindrical cores 6 inches in diameter are used, and for walls from 6 to 10 inches in thickness, 3″ × 6″ elliptical cylinders are employed. These cores are made either 12 or 24 inches in length and have a cross-bar *c* at the top with dowel pins that will fit in holes in the flanges

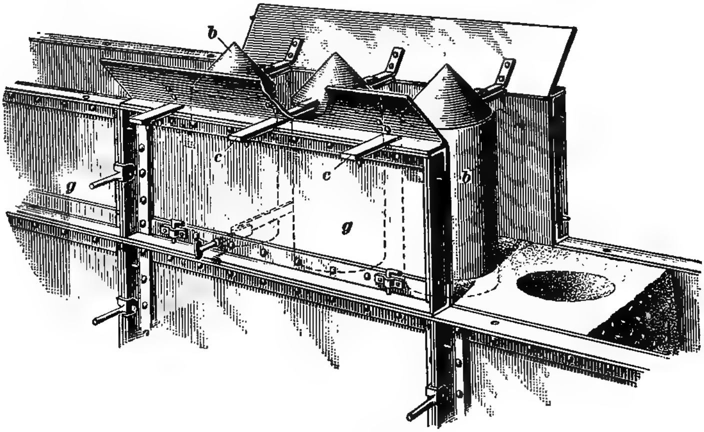


Fig. 36

of the side forms. The forms are held together by an ingenious clamping device *d*, which consists of a fork-like arrangement with a dowel, or pivot, that can be inserted through holes in the flanges of the side forms. When turned with the handle in a horizontal position, the flanges are caught and clamped together by the prong and flange.

To prevent the forms from spreading they are held together by a special tie *e* illustrated in detail in Fig. 38. As will be seen, the tie consists of bar iron bent at the ends, as at *a*, and threaded for a thumbscrew *b*. These ties are left in the concrete work and are not exposed, because, on account of the length of the thumbscrews, the ends are some distance from the surface of the concrete work.

The general method of using the forms, as well as that of supporting the cores with the cross-bars at the top, is illustrated in Fig. 36. The sides of the forms are shown at *g*, and the cylindrical cores to form the hollow spaces in the walls, at *b*, being suspended within the forms from the cross-bars *c*.

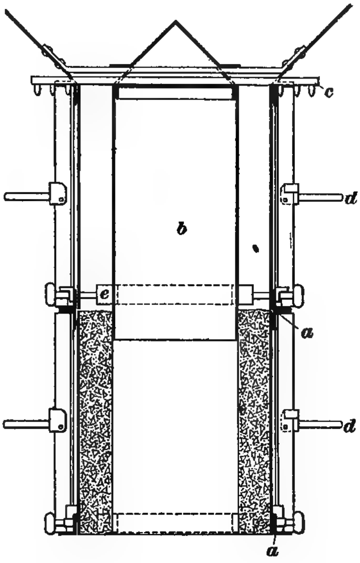


FIG. 37

48. The advantages claimed for the monolith concrete form are that little skilled labor is required to set it up and that it can be conveniently and quickly handled, giving a straight, strong wall that is hollow, dry, and a good non-conductor of heat. Owing to the fact that these forms are made of metal, they produce a wall of smooth surface and with very few joints showing. They can be used with a wet mixture of concrete and may be used over and over again. Inasmuch as they cast a hollow wall, a minimum amount of material is used and the concrete sets up with great rapidity.

FIG. 33

49. Spandrel-Wall Forms.—In the molding of spandrel walls of buildings, or that portion of the wall between the lintel of the window below and the sill of the window above, a special type of form construction must be used.

A typical spandrel-wall form is shown in Fig. 39. In this figure the form work is supported by the studs *a* and outlookers *b*.

The several studs are tied together by ledger boards *d*, which are spiked securely to the form work of the piers on each side of the window opening. To brace the top of the outlooker, oblique struts *c* are provided; these are well spiked to the upright studs and the outlookers.

50. In the construction of the spandrel-wall form itself, the usual method of molding concrete walls is employed. Generally, the sheathing used for the side forms consists of dressed plank running horizontally. In the construction here shown, the wall is straight on the inside and molded with a panel on the exterior, as at *r*. The exterior sheathing boards

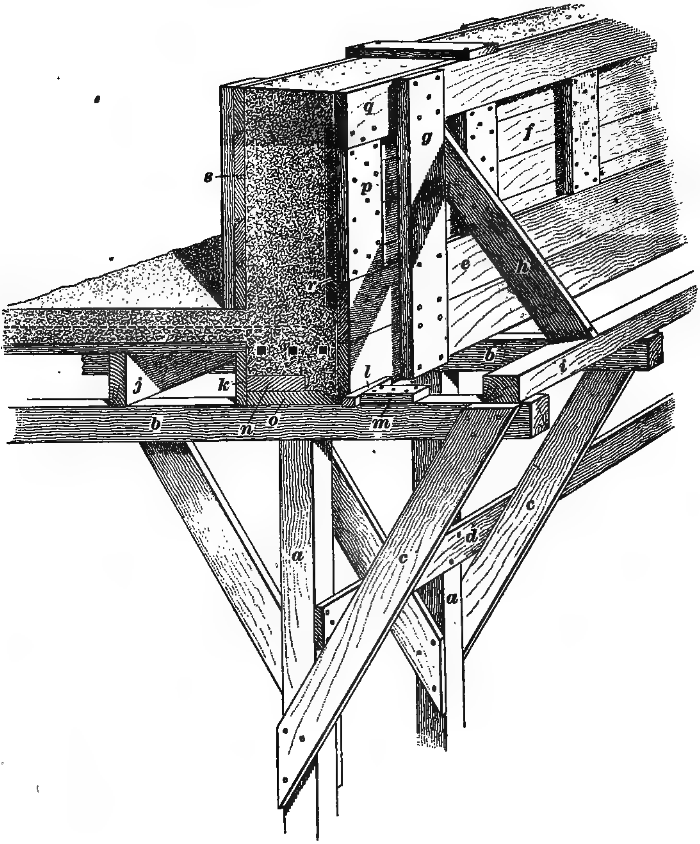


Fig. 39

e and *f*, together with a board *q*, form the profile of the sill, panel, and lintel. The bottom board *o* of the form is usually a 2-inch plank, with a filler piece *n* to form the rabbet for the window frame. The board *k* forms the mold for the inside of the lintel and, together with the joist *j*, acts as a support for the slab centering adjacent to the window head. The spandrel mold is held together and braced by the battens *p*, *s*, and *g*, and in order to keep it in a plumb position is back-braced by a strut *h* notched over the stringer *i*. In order that the bottom of the spandrel-wall form may be straight, a cleat *m* is securely spiked to the outlooker, and by means of the wedge *l*, the bottom board of the form is clamped tightly

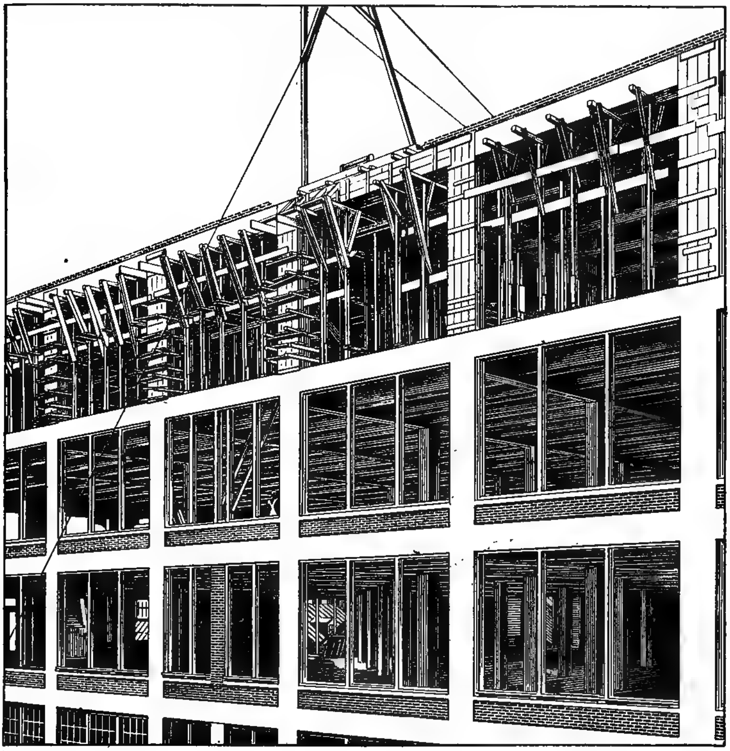


FIG. 40

in position. The spandrel form throughout may be tied together with wire ties or through bolts, in the same manner as any wall.

51. Attention is called to the fact that the spandrel wall cannot be carried up at the time that the lintel and slab forms are filled, especially if wet concrete is used. Therefore, the concrete must first be deposited to the level of the top of the slab, and when this has its initial set, the balance of the wall may be filled. If this precaution is not taken, the hydrostatic pressure of the wet concrete in the spandrel wall will force the concrete from underneath the form at the slab and prevent the filling of the wall form.

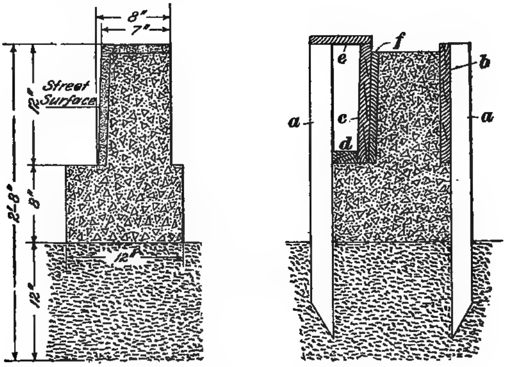


Fig. 41

The centering just described is typical of spandrel-wall construction for concrete buildings, and its general appearance, without paneling, when in place is shown in Fig. 40.

52. Form Work for Curb Construction.—As a curb is simply a low wall, the construction of the forms for casting curbs in concrete will be treated here. In the building of concrete curbs, it is desirable to use as little form work as possible; therefore, a construction similar to that shown in Fig. 41, which is economical in regard to both labor and material, will be found suitable. Wooden forms are provided for only the upper portion of the curb, as the sides of the excavation furnish the form for the concrete of the lower portion.

In the form construction shown in the figure, stakes *a*, which are made of 2″×3″ stuff and properly sharpened, are driven into the cinders or earth upon which the curb is to be constructed. They are spaced about 3 feet apart, and act as the side supports for the planking that makes up the form of the curb above grade. The inside plank *b* is set vertical, but the plank *c* forming the street side of the curb is usually placed out of plumb. This plank is held at the proper position by means of blocking pieces, as at *d*, and cleats *e* nailed to the top of the planks and stakes. In order that the street face of the curb may be finished in a substantial manner, a ¾-inch board *f* is placed on the inside of the form. This board is removed when the concrete in the mass of the curb has set sufficiently, and the space between the concrete and the outside form board *d* is then filled in with finishing mortar. The top finish of the curb is put on at the time as the face of the curb, the form boards being made to project ¾ inch above the concrete base so as to form a gauge for this purpose.

FORMS FOR SPECIAL PURPOSES

MOLDING OF ARCHITECTURAL MEMBERS

53. Imitation Joints.—In buildings whose walls are entirely of concrete an effort is often made to imitate stonework by forming joints in the surface of the concrete. This is done for two reasons: (1) to provide deep shadow lines that will hide the line of demarcation caused by stopping the concrete work in the successive layers, and (2) to break up the surface of the concrete so that discoloration and roughness in molding will not be so noticeable.

The joints may be formed by beveled strips that are fastened upon the face of the form board, as shown in Fig. 42. Of the three strips shown, the one at *a* is the most practical, as it permits the form boards to be readily removed from the face of the concrete. While the strip *b* is often used, it does not pull away from the surface of the concrete so readily as does the beveled strip *a*. This is also true of the strip *c*, and, in addition, the shadow cast by this form of grooving, or joining, is not so pleasing.

54. Forms for the Casting of Undercuts.—Although it is not customary in concrete work to design moldings with undercuts, this kind of work can be done. In Fig. 43 is shown the form work for casting a sill with an undercut drip *a*. In all instances, the best plan would be to cut this drip in the concrete after it has set, but if the drip must be cast, the forms can be arranged as shown. The molded board *e*, which forms the undercut, is arranged so that it will remain in place when the boards *b*, *d*, and *c* that form the face of the sill and the panel of the spandrel are withdrawn. After these form boards have been taken away, the piece *e* can be drawn downwards in the direction of the arrow, thus pulling away from the undercut *a*. It is extremely difficult to do such work as this satisfactorily, and for this reason undercuts should be avoided wherever possible.

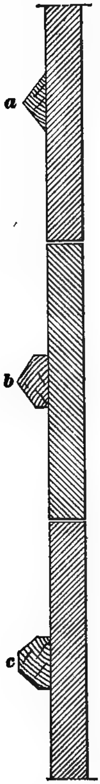


Fig. 42

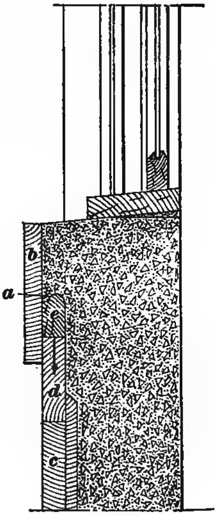


Fig. 43

55. Cornice Construction.—In Fig. 44 is shown a typical form construction for a reinforced-concrete cornice. In the way that this form is constructed, the general type of centering used for spandrel walls is employed. The supports consist of studs *a* and braces *b* securely spiked to the sides of the horizontal outlooker *v*. Upon the outer end of the outlooker is erected a vertical stud *c* and a stud *d*, the two being braced by an oblique piece *f*. The stud *d* is halved out at the top so as to support a stringer *n*, and the outer stud *c* supports the stringer *m*, the two studs *d* and *c* being tied horizontally by the ledger board *g*. The inside of the form for the cornice is back-braced by the oblique member, as shown.

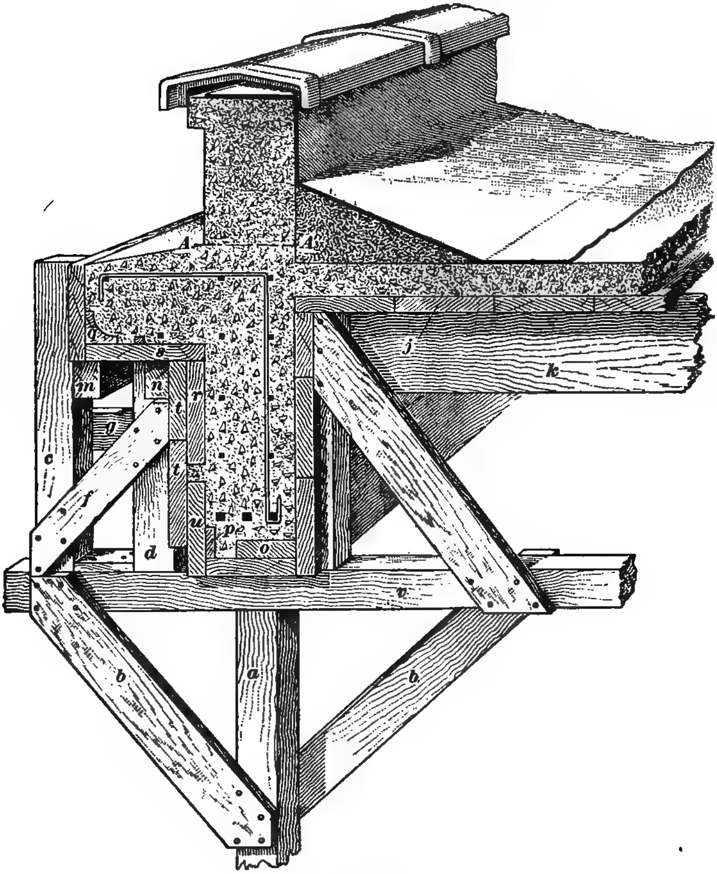


FIG. 44

The entire cornice work is carefully constructed of $1\frac{3}{8}$-inch dressed material, and consists of the several pieces indicated at *p*, *u*, *t*, *s*, etc., together with the molded piece *q*. The pieces *s* and *r* are securely spiked together and held to the two pieces marked *t*. The piece *o* forms the rabbet for the top-story window frame. The roof-beam frame work is shown at *k*, and the roof-slab centering at *j*. In filling the form, the concrete is carried to the level *A A*, and the parapet wall is afterwards constructed above this point. The details of the roof construction are shown in order to illustrate the position of the cornice clearly.

56. Construction of Curved Work.—In the molding of the concrete work for an all-concrete building, curved brackets, fasciæ, and balconies are frequently encountered. Two methods are usually employed in the construction of such work. In one, lagging is nailed to form boards sawed to the required radius, and in the other the form boards are kerfed so that they can be bent to the required form against or around brace blocks.

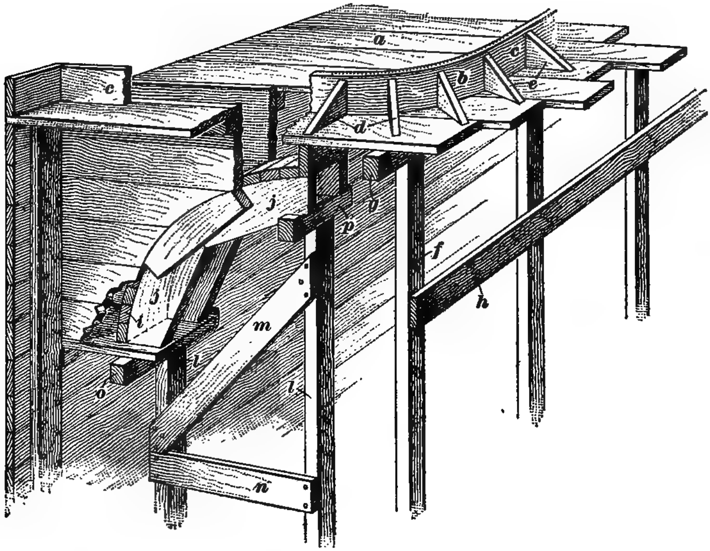


FIG. 45

In Fig. 45 is shown the centering for a balcony with a curved fascia supported upon concrete brackets. This example of centering illustrates both methods of forming curved work.

The bracket is constructed by nailing lagging to curved forms, or centering, and the curved fascia of the balcony is formed by kerfing the board of the form and nailing it to brace blocks. In the illustration, the slab centering of the balcony floor is shown at *a*, and the kerfed form board for the fascia of the balcony at *b*, the board being sawed partly through at intervals sufficiently close to allow it to bend around the required radius. The straight sides of the form are shown at *c*, and the brace blocks at *d* and *e*. The outer edge of the balcony centering is supported by studding *f*, with a plate, or capping piece, *g*, the studding being braced by the ledger board *h*.

In the construction of the form for the bracket, the form planks *j* are sawed to the required radius and spliced by a nailing piece at the several junctions. Upon these forms the lagging *i* is nailed, and the side planking of the bracket is usually run past the ends of the lagging and spiked to it. The form work for the bracket is supported by the studs *l*, with the cap pieces *o* and *p* properly braced by the oblique brace *m* and the tie *n*.

As a rule, light, narrow material is used for the lagging, and it is best in the kerfed work to use light material also, with the pitch, or brace, blocks at close intervals. The forms *j* upon which the lagging is to be fastened should be of 2-inch plank; otherwise, they are liable to split when the lagging is nailed to them.

SPECIAL COLUMN FORMS

57. Wooden Forms for Cylindrical Columns. Specially constructed forms are required for the casting of cylindrical concrete columns. Usually, a type of centering similar to that shown in Fig. 46 is used. In this figure, plank forms *a* are sawed out to the required radius, and provided with battens *c* to prevent splitting. To these, staves *b* are nailed and driven up tight. These staves are usually beveled on the edge, but are not curved, as the necessary cylindrical appearance is gained by making them narrow—not more than about 2 inches in width for 24-inch columns. These forms are made in halves and are securely clamped together by means of yokes *f* and tie-bolts *g*, the yokes having slots *i* at the ends to aid in the placing of the bolts and to allow for some increase in the size of the columns. Frequently, the column forms, as well as the lagging, are bound at intervals by hoop iron ties *h*. These ties are made in halves and are punched on the ends so that they can be bolted together. Sometimes, however, they are wrapped around the lagging several times and secured with wire nails.

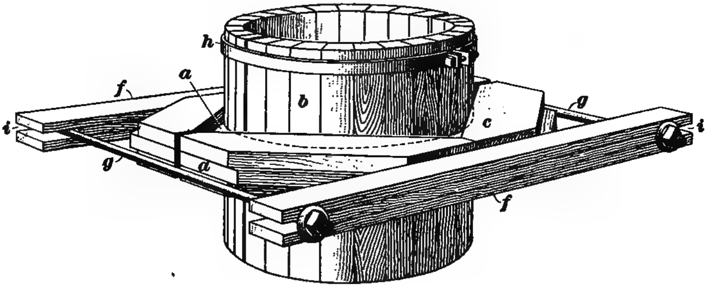


FIG. 46

58. Sheet-Metal Forms for Cylindrical Columns. If cylindrical concrete columns of very smooth appearance are required, sheet-metal forms of the type shown in Fig. 47 can be used. Such forms are generally constructed of about No. 16 tank steel. The steel is rolled to the required radius

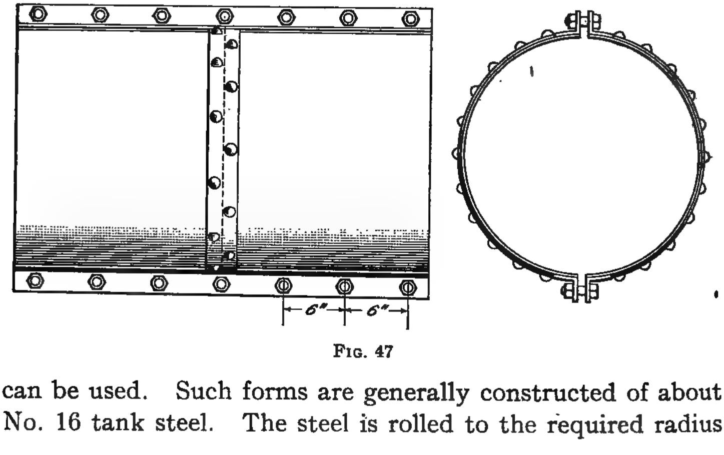
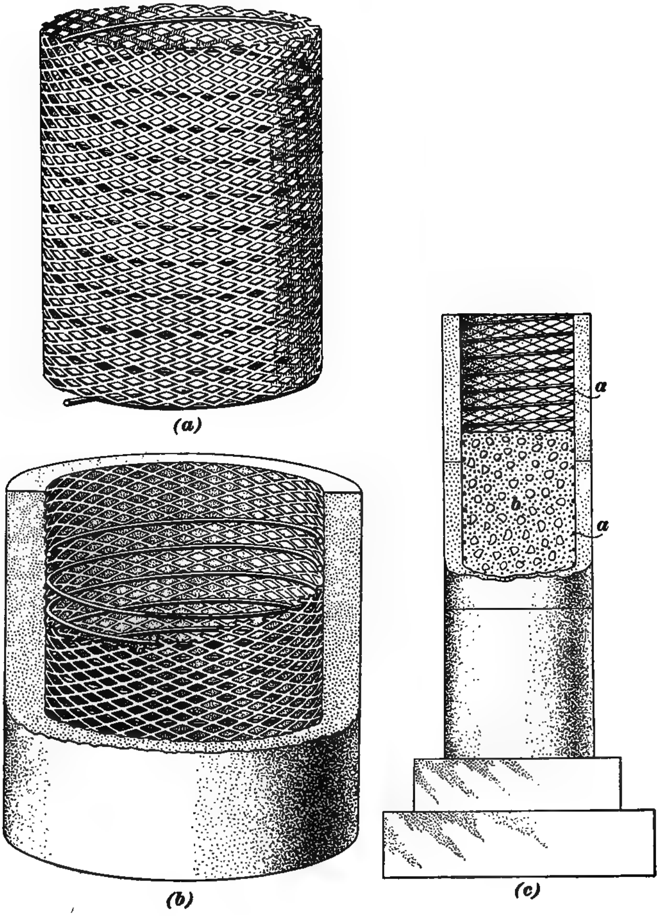


FIG. 47

and formed in halves, with angle irons along the edge for securing the halves together. The several sheets are butted at the ends and are held together with a single butt strap on the outside, the rivets being either flattened or countersunk on the inside of the form.



(a) (b) (c)

Fig. 48

Sheet-metal forms of this kind produce very smooth and true work, though they are much more expensive than wooden forms. When they are to be used, it is well to make the columns throughout the building of one size, so that several sets of forms will serve for the entire construction.

When metallic forms are used, some difficulty is usually encountered in casting the work at the top of the form. At this place, the wooden forms of the beams and girders intersect, and a special collar piece must be provided to fit over the steel form. Studs are also placed close to the metallic form in order to give additional support to the ends of the beam and girder forms.

59. Concrete Forms for Cylindrical Columns. Forms made of concrete reinforced with hoops of steel or with metallic lath are sometimes used in the construction of columns. One of the latest of these forms is illustrated in Fig. 48. In this figure, the cylinders *a*, as shown in view (*c*), are formed of expanded metal wired to hoop reinforcement. In view (*a*) is shown one of these cylinders made of metallic lath which is plastered with cement mortar and smoothed true and to a cylindrical form. When this has set, a reinforced-concrete cylinder strongly hooped, as illustrated in the detail shown at (*b*), is obtained. These cylinders of concrete are placed on top of each other to make the column forms. They are then filled with the concrete, which forms the main core of the column, as shown at *b*, in view (*c*).

The advantage in this type of form work is that the reinforcement is embedded with the actual form construction, and this, being of cement, remains permanently in place and insures a smooth finish for the exterior of the columns.

FORM WORK FOR CONDUITS, CULVERTS, AND STACKS

60. Wooden Conduit Forms.—In the construction of concrete sewers and conduits of large size, it is customary to use a heavily constructed wooden sectional centering. A typical centering for forming the inside of a concrete conduit is shown in Fig. 49. The entire centering consists of seven sections, *a*, *a*, *b*, *b*, *c*, *c*, and *d*. These sections are made of 2- or 3-inch plank sawed to the required shape, with 1-inch lagging *e* on the outside.

The several sections are held in their relative positions and braced by means of 2″×8″ cross-braces *f* and *g*, the top brace *g* supporting the top section *d* of the form. This top section really acts as a key for the entire form construction. When it, together with the cross-brace *g* is removed, the entire form work may be taken down, or collapsed, and moved to its new position for successive construction.

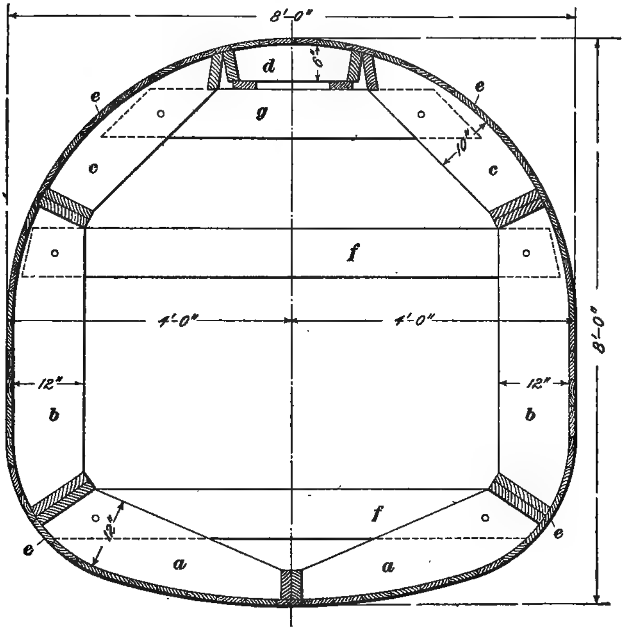


Fig. 49

61. Collapsible Steel Centering.—In Fig. 50 is shown a type of collapsible centering extensively used in some localities for the construction of concrete conduits or sewers. As shown, it consists of two sections, or halves *a* and *b*, of sheet iron. The upper section is arranged with angles, or clips, *c* along the edge so as to make junction with the lower section. These sections are held in their respective positions by means of iron rod or bar braces and turnbuckles. By screwing up the turnbuckles, the ends of the upper half of the centering are drawn in, allowing it to drop within the lower half, thus making a collapsible form.

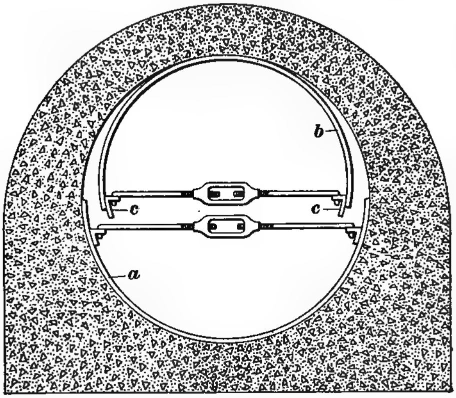


Fig. 50

62. A type of centering known as the **Blaw collapsible conduit or sewer centering** is shown in Fig. 51. It is used in the construction of curved conduits and sewers, and is operated in somewhat the same manner as the centering shown in Fig. 50.

63. Forms for Concrete Chimneys.—In the construction of reinforced-concrete chimneys, several types of centering are employed. In some cases, the construction of the centering is the basis of the working patents of the chimney construction company. The centering may be made of wood, or it may be made of sheet steel braced with angle iron and arranged to bolt together in sections.

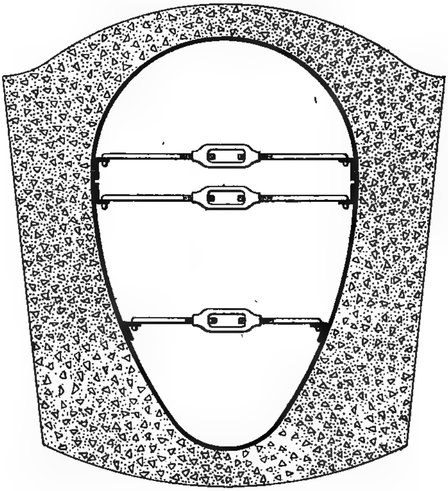


Fig. 51

For the proper construction of the reinforced-concrete chimney there must be an inside and an outside centering, and in building the chimney, the portion of the concrete placed in one day acts as the support for the centering to form the next section. In conjunction with the centering, false work is usually provided for spacing the reinforcing rods, this centering, or spacing, device being arranged several feet above the form work, so that the rods will be held plumb and in their true position. With chimneys of any size, the concrete and materials of construction are hoisted up inside of the chimney as the work progresses.

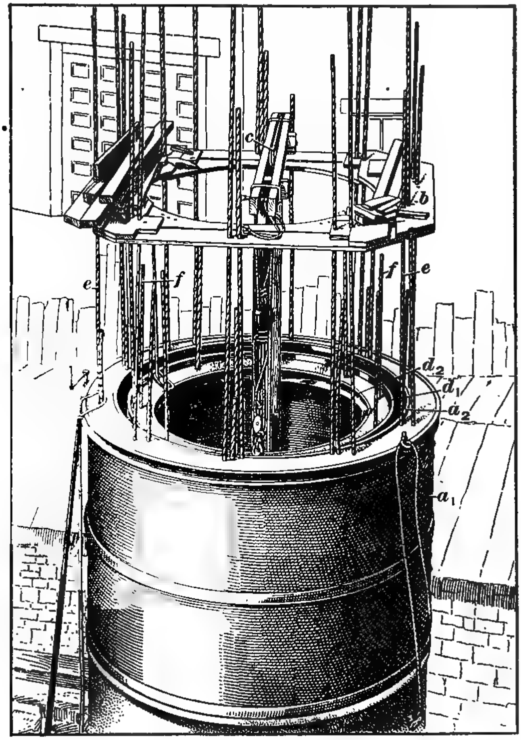


Fig. 52

64. The stack shown in Fig. 52 consists in reality of two separate stacks, an outer and an inner one, the inner one being called the *lining*. In the illustration, the outside and inside steel shells for molding the stack proper are shown at a_1 and a_2, respectively, while the reinforcing rods are shown at e and the

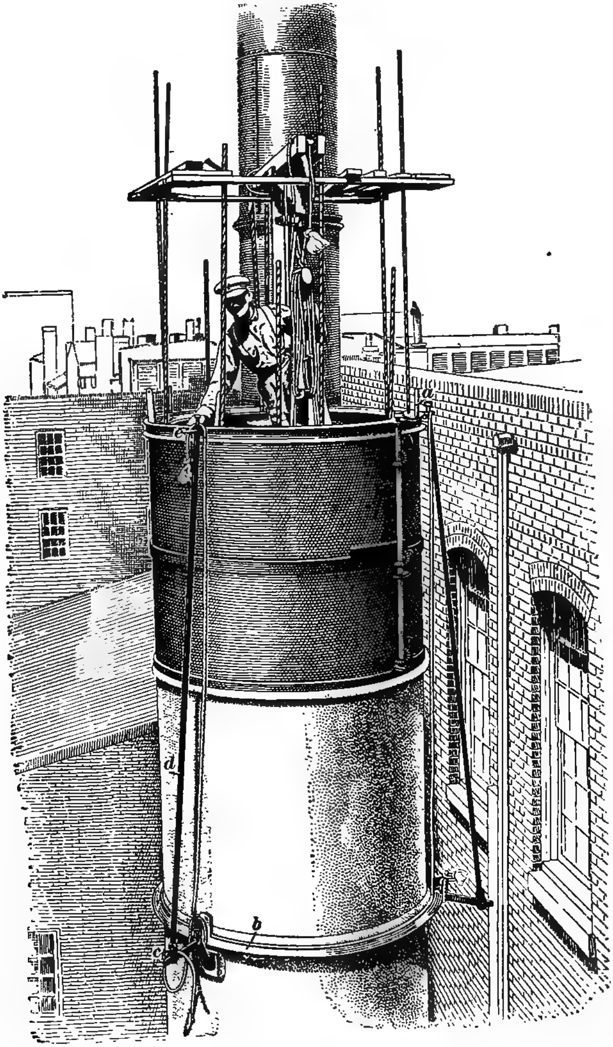


FIG. 53

wooden, or false, work by means of which the rods are placed and held in position, at *b*. The lining is molded by the shells d_1 and d_2 and has reinforcing rods *f*. The hoist for the concrete is arranged inside the stack, the head of the hoist being shown at *c*. The forms are shown already filled with concrete.

After the concrete, which is placed very dry, has set for 10 or 12 hours, the steel cylinders are loosened and raised to an upper position, preparatory to placing another course of concrete.

65. The construction of the sheet-metal stack form shown in Fig. 52 and the method of operating it are better illustrated in Fig. 53. The outer and inner shells are made in halves, which are hinged along one edge and are fitted at the other edge with two angles that are riveted in place. One of these angles is tapped for threaded bolts, which pass through free holes in the other angle. These bolts are equipped with beveled gears and are operated by a shaft, as shown at *a*. By turning the shaft with a crank, the bolts will draw the angles together or release them, causing the cylinders to be decreased or increased in size, so as to free them from the concrete already cast and thus allow them to be moved up in a new position.

As will be observed from the illustration, the cylinders are stiffened at the top and bottom with channels and intermediately with a bar-iron ring. They are held in position for filling with concrete by an ingenious device, as shown at *b*, *c*, *d*, and *e*. Some distance below the stack is a loose angle-iron ring *b*, and to this ring is hinged with bolts a bent lever *c*, or rocker-arm. This arm carries on the vertical leg a wooden friction block, and the other end of the arm is secured with a bolt to the connecting rod *d* running to the top of the steel form. In operating the device, the sheet-steel form is raised so as to lap the top of the finished concrete by about 3 inches, after which the loose angle-iron ring *b* is hoisted up and then held with tackle while the weight of the shell is left to settle back on the connecting rods *d*. This throws the horizontal lever of the rocker arm down, and the wooden friction blocks on the arm clamp tight against the finished concrete. The connecting rods thus hold the form in position, and it may be leveled or adjusted slightly by means of the nuts *e* at the upper end.

Generally, only from 4 to 5 feet of reinforced-concrete chimney is built in a day, this being the height of one section of the steel form.

CENTERING FOR CONCRETE BRIDGE CONSTRUCTION

66. General Requirements.—The construction of the centering and false work for reinforced-concrete bridges does not differ materially from the centering construction for masonry arches. In general, the false work must be sufficiently strong to carry the weight of the wet concrete of the completed structure without appreciable deflection or movement. The face of the forms for the soffit and side walls of the spandrels must be smooth and true in order that the finished concrete work will present a good appearance. All this form work must be securely braced against failure from the pressure of the concrete. The principal requirement, however, is that the centering be so arranged that it may be readily dropped away when the concrete has set sufficiently to sustain its own weight.

67. Size of Timbers.—The size of the false work and timbers for the construction of the centering for reinforced-concrete bridges must be carefully figured for the strains and stresses to which they will be subjected from the weight of the concrete. Generally, the uprights of the bents are built of heavy 12″×12″, 10″×10″, or 8″×10″ timbers, which are securely cross-braced in both directions by 2″×8″, 2″×10″, 3″×10″, or 4″×6″ pieces. The form of the soffit of the arch is obtained by using plank 2 or 3 inches in thickness. These are sawed to the required radius and nailed together, sometimes in several thicknesses, the different pieces breaking joints.

68. Lagging.—The lagging, or sheathing, for the centering of concrete bridges usually consists of 1½- or 2-inch dressed plank laid with straight beveled joints and well driven up and securely spiked. Sometimes, however, rough planking is used, in which case the surface is obtained by plastering the top with either clay or cement and covering this with either canvas or paper. Frequently, two thicknesses of 1-inch tongued-and-grooved flooring boards are used for the lagging, or else a rough lagging covered with 1-inch dressed material is used. In all instances, the lagging must be smooth enough to permit the centering to be readily removed from the concrete. Sometimes the wooden lagging is covered with paper to prevent it from adhering to the concrete, and in nearly all instances, the lagging is soaped or oiled to facilitate its removal.

69. Methods of Dropping Centering.—In the design of centering for reinforced-concrete arches, some means of dropping the centering uniformly and gradually must be provided. Probably the best method is to use under the main uprights, or under the false work supporting the lagging, heavy double wedges of hardwood that can be backed out and thus allow the centering to settle. Another method is to use what is known as a *sand box*, which consists of a tight metal box fitted with a plunger upon which the uprights stand. The box is filled with sand, and in the box is provided a small aperture that can be closed with a screw plug or in some other way. To drop the centering, the aperture is opened and the sand is gradually allowed to run out, thus slowly allowing the centering to settle.

Sometimes the centering is constructed on screw jacks, and where several arch rings are to be built parallel with one another, the entire centering is erected upon trucks or rollers built on a track. Thus, when one arch ring has been completed, the false work can be shifted to a new position.

70. Cambering.—In the construction of all false work for arch construction, a slight camber must be given to it, as there will be some settlement in both the timber work and the concrete arch ring. Specifications for the construction of reinforced-concrete arches usually state that the camber must be $\frac{1}{800}$ part of the span greater than the rise called for by the plans.

SPECIFICATIONS AND COST OF REINFORCED CONCRETE

PLAIN- AND REINFORCED-CONCRETE SPECIFICATIONS

REQUIREMENTS AND FORMS OF SPECIFICATIONS

1. Many forms of specifications are used in the erection of reinforced-concrete structures. One form commonly used by architects is extremely brief and very general in its scope. It is sufficiently broad in its requirements, however, to place the entire responsibility for the success and safety of the reinforced-concrete work upon the contractor. In such a specification, the main features of the several stipulations have the aim of attaining certain results, and the contractor is left to accomplish these results by manner and means that to him are most advisable.

Another type of specifications, used particularly by architects and engineers who are specialists in reinforced-concrete construction, treats extensively and in detail upon this construction. It includes a description of the work and takes up item by item the materials, workmanship, method of procedure, and final results to be accomplished. In order to be complete, a specification of this kind must first include the general requirements. These consist of stipulations that cover the furnishing of framing plans by the contractors for the architect's approval, and stipulations requiring the com-

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pliance with the laws and regulations relating to reinforced-concrete work in the locality of the operation. The character of the steel reinforcements should then be treated upon at length, and if several types of reinforcement are to be recognized, the types must be stated and the necessary stipulations regarding each type given. After this logically follows the proportion of the concrete mixture, with stipulations regarding the method of mixing and the precautionary measures to be used when the concrete is placed at extreme temperatures, such as in freezing weather or in very hot weather. The form work and false work would be embodied under another stipulation. This portion of the subject should be carefully treated in order to provide proper material of suitable dimensions and the necessary finish to secure smooth and true work. Precautionary clauses should be inserted regarding the safety of the shoring, or false work, and its time of removal. There should also be included clauses that relate to the placing of the steel reinforcement in the forms and the manner in which it is to be fixed in position so as to prevent it from being disturbed by the tamping of the concrete. The specifications should close with stipulations relating to the patching of the work, the surface finish, the test load to be placed upon the floor systems, and the final cleaning down of the work.

2. As with any other specification, the stipulations for reinforced-concrete work should be clearly and briefly stated, using the nomenclature generally employed in this work. They should also have the stipulations arranged in an orderly manner; thus, the general requirements and instructions should precede the detailed stipulations in the specifications, the body of the specification should cover the nature of the materials and the workmanship, and the final clauses should relate to the finish of the work. When finished, the specifications should in all instances be carefully checked with the plans and detailed drawings, so that there will be no conflict between the stipulations of the specifications and the notes or construction shown on the drawings. The terms of the specifications, especially on reinforced-concrete work, should be moderate, and requirements that cannot readily be enforced or that involve great additional expense over and above the usual commercial method of procedure should not be asked.

In writing and enforcing a specification upon reinforced-concrete work, it must be constantly borne in mind that defects cannot be easily altered after the work has been cast. The clauses of the specifications had better be in the nature of instructions regarding the manner of conducting the work than non-enforcible clauses embodying condemnations of defective finished work.

3. It is difficult to give a form of reinforced-concrete specifications that will fit all conditions pertaining to every piece of work. However, the conditions under which each structure is to be built must be specified concisely. Therefore, four types of reinforced-concrete specifications of different scope and wording will be given here. By making slight changes, any of these specifications may be adopted for specific reinforced-concrete buildings.

The first of these specifications is of a brief content, such as would be included in an architect's specification for a structure in which the reinforced-concrete work is not of great extent or a large percentage of the cost of the work. In some instances, this short form of reinforced-concrete specification could be used where all the floors of the building are of reinforced concrete and where this portion of the work is of the most importance. Its use in such cases is not to be recommended, however, as it is not sufficiently explicit and precautionary. It is adequate, though, for small works or in cases where the extent of the reinforced-concrete work is limited and does not represent the main feature of the construction.

The second form given includes an extended specification for the Portland cement and other materials of the concrete. It also covers both plain- and reinforced-concrete work and other details of construction that depend on this work.

The two longer forms of reinforced-concrete specifications cover entirely the materials and the workmanship involved in the construction. If properly enforced, these stipulations should secure excellent work erected under safe conditions and at a reasonable cost.

In each of these specifications, corresponding clauses are worded differently so as to give a choice in expression and to illustrate as far as possible the method followed in writing reinforced-concrete specifications.

CONDENSED SPECIFICATION FOR REINFORCED-CONCRETE WORK

4. The following specification is the briefest form that can be used to specify reinforced-concrete construction. It states in a concise manner the requirements of the architect regarding the submission of the construction for approval, the character of the concrete and the steel reinforcement, and the floor load upon which the calculation for the design of the floor system is based.

REINFORCED-CONCRETE CONSTRUCTION

5. General.—The reinforced-concrete construction shall be of an approved and successful type, and the contractor shall submit for the architect's approval framing plans and details of construction, together with schedules showing the amount of steel reinforcement in all the beams, girders, and columns.

The contractor is to assume all responsibility for the safety and protection of the work during construction, and shall deliver the same in a complete and finished condition in compliance with the specifications and the accompanying plans.

6. Concrete.—The concrete for beams, girders, and slabs shall consist of 1 part of Portland cement, $2\frac{1}{2}$ parts of sand, and 5 parts of broken stone or gravel. The concrete for columns shall be a 1–2–4 mixture.

7. Cement.—The cement shall conform to the requirements of the standard specifications for cement adopted by The American Society for Testing Materials on November 14, 1904.

8. Aggregates.—The sand shall be clean, and shall not contain over 3 per cent. of loam. The broken stone and gravel shall be hard and close-grained and free from dust and dirt; they shall be of such size as to pass through a ring 1 inch in diameter.

9. Reinforcing Steel.—The steel for the reinforcement shall be manufactured by the open-hearth process. This steel shall have an ultimate tensile strength of from 60,000 to 70,000 pounds per square inch and an elastic limit of at least half that amount, with an elongation of at least 20 per cent. A bar shall bend cold through an angle of 180° and close down on itself without showing signs of fracture.

The beam and girder reinforcement shall consist of square twisted or some other approved type of deformed bar, and the slab reinforcement shall be of square twisted deformed bars or of an approved form of expanded metal or woven or electrically welded wire fabric. The column rods shall be of plain round or square twisted bars. The girders and beams must have the rods so bent and arranged as to provide amply for the bending moments throughout their length and also to take care of the negative bending moments near the bearings and supports. Sufficient stirrups must be provided near the bearings or throughout the length of the beams and girders to resist the horizontal and oblique shearing and tensile stresses.

10. Floor and Test Loads.—The floors and columns throughout shall be designed to sustain a live floor load of 150 pounds per square foot. The finished floor systems shall be tested in two places, to be designated. The test load shall be equal to one and one-half the amount of the live floor load. The test loads shall be so placed as to load fully the girders and beams of one bay.

SPECIFICATION INCLUDING PLAIN- AND REINFORCED-CONCRETE CONSTRUCTION

11. The specifications that follow are prepared for governmental buildings and include matter relating to the cement and the other ingredients of the concrete. They are interesting because they relate to a reinforced-concrete floor construction placed upon steel beams and girders, which are fireproofed with concrete. They include, besides, an alternate specification for an all-reinforced-concrete floor system. The portion relating to the materials for the concrete is especially complete and represents governmental requirements. Where the term "soft material" is used, it means a material that serves more the purpose of a filling or weight than one that has to resist a stress.

PLAIN- AND REINFORCED-CONCRETE WORK—CEMENT AND CONCRETE

12. General.—All cement used for concrete must be true Portland cement, made from natural rock without the use of furnace slag in any form, whether it is calcined after grinding and mixing with lime or not. Cement for concrete purposes must comply with the following requirements:

The average weight per barrel shall not be less than 375 pounds, net. Four sacks shall contain 1 barrel of cement. If the weight, as determined by test weighings, is found to be below 375 pounds per barrel, the cement may be rejected.

Tests may be made of the fineness, specific gravity, soundness, time of setting, and tensile strength of the cement.

13. Fineness.—Ninety-two per cent. of the cement must pass through a sieve made of No. 40 wire, Stubb's gauge, having 10,000 openings per square inch.

14. Specific Gravity.—The specific gravity of the cement, as determined from a sample that has been carefully dried, shall be between 3.10 and 3.25.

15. Soundness.—To test the soundness of the cement, at least 2 parts of neat cement mixed for 5 minutes with 20 per cent. of water, by weight, shall be made into two pats on glass. Each pat shall be about 3 inches in diameter and $\frac{1}{2}$ inch thick at the center, tapering thence to a thin edge. The pats are to be kept under a wet cloth until finally set, when one is to be placed in fresh water for 28 days. The second pat will be placed in water that will be raised to the boiling point for 6 hours and then allowed to cool. Neither pat should show distortion or cracks. The boiling test may or may not be rejected, at the option of the architect or engineer.

16. Time of Setting.—The cement shall not acquire its initial set in less than 45 minutes, and must have acquired its final set in 10 hours.

The pats made to test the soundness may be used in determining the time of setting. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a $\frac{1}{12}$-inch wire loaded to weigh $\frac{1}{4}$ pound. The final set has been acquired when the pat will bear, without being appreciably indented, a $\frac{1}{24}$-inch wire loaded to weigh 1 pound.

17. Tensile Strength and Test Requirements. Briquets made of neat cement, after being kept in air for 24 hours under a wet cloth and the balance of the time in water, shall develop tensile strength per square inch as follows: After 7 days, 400 pounds; after 28 days, 600 pounds.

Briquets made of 1 part of cement and 3 parts of standard sand, by weight, shall develop tensile strength per square inch as follows: After 7 days, 200 pounds; after 28 days, 275 pounds.

The highest result from each set of briquets made at any one time is to be considered the governing test. Any cement not showing greater strength in the 28-day tests than in the 7-day tests will be rejected.

18. When making briquets, neat cement shall be mixed with from 20 to 25 per cent. of water, by weight. After being thoroughly mixed and worked for 5 minutes, the cement, or mortar, shall be placed in the briquet mold in four equal layers, and each layer rammed and compressed by thirty blows of a soft brass or copper rammer $\frac{3}{4}$ inch in diameter (or .7 inch square, with rounded corners), weighing 1 pound. It is to be allowed to drop on the mixture from a height of about $\frac{1}{2}$ inch. After the ramming has been completed, the surplus cement shall be struck off and the final layer smoothed with a trowel held almost horizontal and drawn back with sufficient pressure to make its edge follow the surface of the mold. The method of compacting the briquets in the molds may be varied, provided a method that is equally as efficient as that specified is adopted.

19. The above are to be considered the minimum requirements. Unless a cement has an established reputation for excellence, the contractor must deliver a sample barrel for test before beginning work. If this sample shows higher tests than those given above, the average of tests made on subsequent shipments must come up to those found with the samples.

20. A cement may be rejected if it fails to meet any of the above requirements. An agent of the contractor may be present at the making of the tests, or, in the case of the failure of any of them, he may require them to be repeated in his presence. All tests must be made by some person of recognized skill, who must be satisfactory to the architect or engineer. All expenses of such tests are to be paid by the contractor. All such tests shall be made on samples furnished from cement actually delivered for use on the work. One sample must be taken from every 10 barrels, unless a smaller number be specifically authorized by the architect or engineer.

21. Delivery.—The cement must be delivered to the site in the original packages, with the brand and the maker's name plainly marked thereon, and the cement must be kept dry, ready for use.

The contractor must, at an early stage of the work, erect at a point to be approved, a cement shed capable of holding at least 2 weeks' supply of cement. The cement must be delivered to this shed and the 7-day test must be completed before it is used.

The contractor shall furnish to the architect or engineer semimonthly statements, certified to by the contractor and the dealer furnishing the cement, showing the amounts and kinds furnished and delivered.

22. Proportion of Cement.—In the proportioning of concrete, 375 pounds of cement, net weight, will be assumed to be $\frac{1}{7}$ cubic yard in volume.

23. Sand for Concrete.—The sand must be a coarse, clean bank sand, screened to remove pebbles larger than a pea, and free from loam or clay. Sand that has been dredged from salt water or bank sand washed with salt water will not be accepted.

24. Stone for Concrete.—Broken stone, clean gravel, or broken bricks, depending on circumstances, shall be used in the concrete. Broken stone, if used, must be a clean, hard rock, broken to pass a 2-inch screen. It must be free from fine dust, but otherwise just as it comes from the crusher, including all sizes from that of fine sand up to the 2-inch stone. Gravel, if used, must be perfectly clean and must vary in size from coarse sand or a small pea to not over 2 inches in diameter. Broken brick, if used, must be broken to pass a $1\frac{1}{2}$-inch screen and must be free from fine dust.

25. Boxing for Concrete.—Footings for all walls, piers, etc. are to be laid in plank cribs of the dimensions required. All planks are to be removed before back filling, unless, in special cases, the architect or engineer permits them to remain.

26. If the soil will permit of neat excavation, planking may be omitted upon the written approval of the architect or engineer; but in such cases any concrete that is damaged by earth caving in on it must be taken out and replaced by the contractor at his own expense.

27. Composition of Concrete.—For footings under piers, columns, walls, etc., concrete must be mixed in the proportion of 6 parts of broken stone or gravel, $2\frac{1}{2}$ parts of sand, and 1 part of Portland cement. All stone and sand must be actually measured. The cement must be weighed from time to time to determine whether or not the barrels are of full net weight. Concrete put in to take the place of soft material or used as a foundation for a cement or asphalt floor, may be mixed with broken bricks, broken stone, or gravel. Where used for the foundation of a floor, it must be mixed as above prescribed for footings. Where used to take the place of soft material, it must be mixed in the proportion of 10 parts of broken stone, gravel, or brick to 4 parts of sand and 1 part of Portland cement.

28. Machine Mixing.—All concrete may be mixed by any power-driven machine that mixes a batch at a time and produces a result satisfactory to the architect or engineer. Continuous mixers will not be allowed except by permission of the engineer. If the contractor prefers, he may mix by hand, as follows:

Sand and cement of qualities elsewhere specified, to be mixed dry on special platforms or shallow boxes, being first accurately measured in the proportions specified. No "guesswork" will be allowed. The proper quantity of water is to be added by means of a hose sprinkler, and the material is to be thoroughly mixed to a stiff paste. The broken stone is to be drenched and drained, placed upon the cement mixture, and turned over sufficiently to incorporate the whole mass thoroughly, but not less than four times. All parts of stone are to be coated with the cement mortar, and all parts of mortar are to be wet, but only sufficient water is to be used to accomplish such result.

29. Tamping of Concrete.—All concrete must be placed in position immediately after mixing. It must be thrown into the trench, or box, from a level not more than 6 feet above the bottom and be at once tamped solidly in place, in layers not exceeding 12 inches in thickness. The maul used for this purpose shall be heavy enough to secure perfect consolidation, and, in order not to break small pieces of the aggregate in the process of ramming, it shall have the striking surface made of hardwood cut across the grain. The work on any unit of concrete must be as nearly continuous as possible, so as to make the work monolithic in character. The concrete must be placed in position wet enough to cause decided quaking after vigorous ramming. If caisson work be used, special means for filling the same must be adopted; the method adopted must be approved by the architect or engineer.

30. Retempering of Concrete.—No wet or moistened mixture containing cement shall remain unplaced for a period exceeding 30 minutes, and it must not be used at all after its initial set. Any such mixture having its initial set before being placed must be removed from the premises or used as dirt filling.

31. Layers to Be Swept.—The concrete work that is being built up in layers is to be swept clean with brooms, sprinkled with a hose, and, if considered necessary by the architect or engineer, made rough by means of a pick before adding the next layer. Where a good bond is essential, the surface must also be covered to the depth of 1 inch with a mortar consisting of 1 part of coarse sand and 1 part of Portland cement.

32. Top Surface.—The top surfaces of concrete footings, when finished, shall be brought to a smooth even surface, and shall be finished with cement mortar, if necessary, to bring them to such a surface. The cement mortar is to be mixed in the proportions named above for mixture of cement and sand.

REINFORCED-CONCRETE FLOOR CONSTRUCTION

33. General.—Consideration will be given to a floor system of concrete-slab construction carried on steel beams, and as hereinafter specified. Proposals will also be taken on a system of reinforced-concrete construction without the general use of steel beams, as shown and hereinafter specified. Lump-sum bids are to include the former system.

34. The under side of all floors showing in the story below are to be finished rough for plastering, unless otherwise specified.

All roofs are to be of same general construction as the floors. The surface of the roof construction is to be shaped and graded with the proper pitch, gutters are to be formed and sloped to conductors, and the top of the concrete work is to be brought to an even surface with uniform grade and without holes, ready for the laying of roofing.

35. Floor Slabs.—The floor slab for the layout of steel beams shown shall be of concrete with a total thickness of 4½ inches. For the 7-foot span, it must have steel reinforcement amounting to .52 square inch of steel for every foot in width of the slab. For the 6-foot spans, it must have not less than .42 square inch of steel for each foot in width of the slab. By "width" is meant the dimension parallel to the steel **I** beams. The steel must be placed with its center at a distance of 2¾ inches from top of the slab and must be in the form of small round bars. The slab shall be reinforced in a direction parallel to the steel beams by round steel bars ¼ inch thick, spaced 18 inches on centers.

The reinforcing bars running at right angles to the beams shall be bent so as to rise to about the middle of the slab where they pass over the top of the steel **I** beams. At least every other reinforcing bar must be continuous over the top of every steel **I** beam, except at the walls, where they shall all be cut off.

36. Beam Protection.—The beams are to be protected by concrete, giving a minimum thickness of 2 inches below the flanges, and of 1½ inches sidewise, measured from the edges of the flanges. This covering is to be built solidly into the web of the beam and is to be reinforced by **U**-shaped bars. These bars are to be made of round iron ⅜-inch in diameter and are to be spaced at intervals of about 1 foot. The upper ends of these **U**-shaped bars must be bent over the nearest reinforcing bars in the floor slabs. The lower angles of the

U's must carry longitudinal $\frac{3}{16}$-inch round bars running parallel with the beams. The steel reinforcement of the concrete beam covering is to occupy on the under side a mid-position between the outer surface of the concrete and the surface of the steel beam. The stirrups need not be bent to conform to the profile of the I beam, but may be placed 1 inch up from the soffit of the beam protection and $\frac{3}{4}$ inch in from the sides.

The floor slabs must be spaced high enough to have at least 3 inches of thickness on top of the I beams.

37. Concrete Mixture.—The concrete for the slabs and beam protection described herein is to be formed of cement, sand, and gravel or broken brick, or broken terra cotta. The aggregate may vary in size up to $\frac{3}{4}$ inch, but none is to be used that will fail to pass a $\frac{3}{4}$-inch screen. The slab mixture is to be 1 part of cement, $2\frac{1}{2}$ parts of sand, and 6 parts of gravel or broken brick. That for beam covering is to be 1 part of cement, 2 parts of sand, and 5 parts of gravel or broken brick, these latter being especially small. The aggregates for all concrete is to be subject in every respect to the approval of the architect or engineer

All concrete must be put in position rather wet or plastic, and every precaution must be taken to keep the forms and centers true, so that in finishing it will be necessary only to put on the white coat of plaster.

38. Reinforced Concrete (Alternate).—In lieu of steel beams and concrete floor slabs, consideration will be given to floor systems designed entirely in reinforced concrete. A typical design, using a patented bar, is shown in the accompanying drawings.* There is no prejudice in favor of this particular bar, but the reinforced concrete must in all cases be armored to at least the extent shown in the drawings. It must have inclined members similar to the sheared-up web in the Kahn bar set at an angle of 45° with the horizontal and rigidly fastened to the horizontal bars. These sheared members must be so proportioned that the total horizontal component of the adhesion of all of them in the concrete will be equal to the ultimate strength of the horizontal bar to which they are attached. In lieu of the Kahn bars, plain rectangular bars with drilled holes and round rods put through these holes and bent to form **U**-shaped stirrups will be accepted, but the net section of the rectangular bar must in all cases be equal to that shown for the Kahn bars in the drawings.

39. The contractor must particularly see to it that the owner is preserved harmless against claims for royalty in any reinforced-concrete work.

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*The drawings referred to are the plans of the building that always should accompany the specification.

40. All concrete floor construction must be executed strictly according to the drawings, and the amount of concrete shown as fire protection to the steel reinforcement in the drawing must be regarded as a minimum.

41. All work must be smooth and true when finished. The centering must be made of dressed lumber and must be so accurately set that the exposed surface of the concrete will need only a floated coat of plaster of Paris as a finish. The centering must be rigid enough to prevent noticeable deflection of girders, beams, and floor slabs.

42. The concrete must be made of Portland cement equal to that specified for the work, and coarse, clean sand and clean gravel. The gravel must vary in size from a pea to a sphere 1 inch in diameter. No gravel will be accepted that will not pass a 1-inch screen. The proportions must be 1 part of cement and 2½ parts of sand to 5½ or 6 parts of gravel, according to circumstances. Should it appear advisable to the architect or engineer, he may order reasonable variations in these proportions.

43. Wherever a girder is used, it must be made monolithic with the slab that rests upon it. In order to accomplish this, the filling of the girder and slab forms with concrete must be a continuous operation. In executing the work, a section across the entire building must be put in at one time. Where work is discontinued on one day, the concrete slab must be finished on a line parallel to the reinforcement and extending from outer wall to outer wall. Joints in any other direction between the work of successive days will not be permitted.

44. Arch Filling.—The filling over floor arches or slabs and under all roof gutters where shown to be filled, and forming slope of roof where necessary, is to be of concrete formed of 7 parts of clinkers and ashes to 1 part of cement. The under masonry must be well swept and drenched before laying the concrete filling. The filling is to be thoroughly tamped so as to form a solid mass. Any appreciable percentage of combustible matter will not be allowed. If clinkers free from combustible matter cannot be obtained, broken brick and sand may be used.

45. Where wooden floors are to be laid, the filling must be flush with the top of the wooden sleepers. Where marble, cement, or mosaic floors are shown or specified, the filling is to be an approved mixture containing sufficient additional cement to make a satisfactory and solid bed, and is to be of the proper depth to bring the finished floor up to the required level.

EXTENDED SPECIFICATION FOR REINFORCED-CONCRETE CONSTRUCTION

46. The specification that follows is an extended form of reinforced-concrete specification, in which the detail requirements of the construction are treated at some length. In this specification are included the requirements for the cement, which are sometimes placed under a separate heading. As will be observed, the first two clauses of the specification embody the usual instructions to bidders and relate to the general quality of the work that is expected. They also require that the successful subcontractor shall have had adequate experience in reinforced-concrete construction. The third clause requires that the contractor shall submit detail working drawings for the architect's approval, and that the work shall comply with the requirements of the building laws. Then follows the stipulation for the cement, which includes requirements in regard to quality, protection, inspection, and tests and clauses relating to fineness, set, and maximum proportions of the undesirable chemical constituents.

After the cement specifications naturally follow clauses relating to the sand, broken stone, and proportions of these ingredients in the concrete. Then come, in logical order, statements about the mixing of the concrete, the requirements of the steel reinforcement, and the placing of the concrete, with a clause relating to the placing of concrete in freezing weather. Considerable space is devoted to the wooden forms, their general construction, and the time of removal, with a clause that places the responsibility for their removal on the contractor.

In order that the reinforcement may be intelligently designed, the test requirements for the floor systems are given. These are followed with a clause in which the allowable stresses are stipulated, so that the basis of the calculations for the sizes of the beams, girders, and columns, together with the sectional area of the steel reinforcement, is fixed when the live floor load, which is included in a following clause, is known. The specification finishes with details of construction relating to the finishing and surfacing and stipulations relating to sleeves and sockets. Frequently, the specification for the cement floor of the basement, or ground floor, is embodied in the specification for the reinforced-concrete work, and is here included.

REINFORCED CONCRETE

47. General.—All work included in this specification must be of the best material and workmanship. Strict attention must be given to the selection of materials and experienced labor, and effective and constant supervision must be maintained.

48. Bidders.—Estimates will not be received except from thoroughly experienced contractors in this class of work, who must have constructed work at least equal in strength and character to that required by these drawings and specifications.

49. Detailed Drawings.—The dimensions of beams, columns, floor slabs, reinforcement, and other structural work, as indicated upon drawings, shall be considered as the minimum, but the contractor will be required to furnish shop drawings that show full details of construction, method of calculation, etc. for the approval of architects and the Bureau of Building Inspection, and these must be satisfactory to both. Systems of construction that have not been approved by the building departments of both New York City and Philadelphia will not be permitted. Bids submitted without attention to these details will not be considered.

50. Quality of Cement.—All the cement used in building shall be of the best quality Portland cement and shall be dry and free from lumps and of brands satisfactory to the architects and engineers.

51. Cement Packages.—Cement shall be packed in canvas bags or in barrels lined with paper. Such packages shall be plainly marked with the brand and the name of manufacturer. Bags shall contain 94 pounds, net, and barrels 376 pounds, net.

52. Protection of Cement.—Cement must be stored in a shed in such a manner as to permit easy inspection and identification of each shipment. The floor of the shed must be raised from the ground and properly weather-proofed to protect cement from storm or dampness.

53. Inspection of Cement.—All cement must be inspected. The contractor must submit a sample of cement and provide every facility for inspection and testing at least 12 days before cement is to be used. Architects are to be notified at once upon receipt of each shipment for the work. No cement will be accepted or allowed to be used unless delivered in suitable packages and properly branded. All cement that does not pass tests must be immediately removed from the premises, and sufficient quantity of tested cement must be kept on hand at all times to enable the prosecution of the work without delay due to rejection of new shipments.

All cement shall be inspected and tested by experts satisfactory to the architects and engineers. The cost of all testing is to be paid by contractor. The contractor is to figure on 10 per cent. of all packages of cement being tested, but the architects reserve the right to take samples and test as many more packages as may in their judgment be necessary to establish the character of any particular shipment. Cement failing to meet the 7-day requirement shall be held awaiting the result of the 28-day test before rejection. The acceptance or rejection of cement shall be based on the following requirements:

54. Cement Tests (Neat Cement).—The specific gravity must be not less than 3.1. The ultimate tensile strength per square inch shall not be less than the following:

NEAT CEMENT	POUNDS
24 hours in moist air	200
7 days (1 day in air, 6 days in water)	550
28 days (1 day in air, 27 days in water)	750

CEMENT AND SAND

(1 part cement, 3 parts sand)

7 days (1 day in air, 6 days in water)	200
28 days (1 day in air, 27 days in water)	300

55. Cement Requirements.—The residue on a No. 100 sieve shall not be over 5 per cent., by weight, and on a No. 200 sieve not over 25 per cent., by weight.

These sieves shall be of brass wire cloth having approximately 9,800 and 37,500 meshes per square inch, respectively, the diameters of the wire being .0045 and .0025 inch, respectively.

The cement shall require not less than 30 minutes to develop initial set, but must develop hard set in not less than 1 hour nor more than 10 hours.

56. Constancy of Volume.—Pats of neat cement about 3 inches in diameter and $\frac{1}{2}$ inch thick at the center and tapering to a thin edge shall be kept in moist air for a period of 24 hours. A pat is to be kept in air at normal temperature, and observed at intervals for a period of at least 28 days. Another pat is to be kept in water maintained at 70° F. and observed at intervals for at least 28 days. A third pat is to be exposed in any convenient way in an atmosphere of steam above boiling water in a loosely closed vessel for 5 hours. These pats, to pass requirements satisfactorily, shall remain firm and hard and shall show no sign of distortion, checking, or disintegration.

57. Sulphuric Anhydride, SO_3.—The cement shall not contain more than 1.75 per cent. of sulphuric anhydride and not more than 3.5 per cent. of magnesia, MgO.

58. Cement Briquets.—Briquets for testing shall be 1 square inch area of cross-section.

59. Sand.—Sand shall be clean sand or gravel satisfactory to the architects and engineers. It shall be coarse, with grains graded from coarse to fine, and free from loam, clay, dirt, or other impurities, and with the smallest possible percentage of voids.

60. Broken Stone.—Broken stone shall consist of hard, durable trap rock not exceeding $\frac{3}{4}$ inch in main dimension, and shall be the "run of the crusher," with all dust screened out. No other stone shall be used except with approval of the architects and engineers. No stone not free from argillaceous and organic matter will be accepted.

61. Proportions.—The concrete for all slabs, beams, columns, walls, and all places where concrete is to be used shall consist of 1 part of Portland cement, 2 parts of sand, and 4 parts of broken stone.

62. Mixing. — All concrete shall be machine-mixed, using either a batch or a continuous mixer of approved design. Plenty of clean water, free from all oil, dirt, or impurities, shall be used, so that the result of the mixture will be of such consistency that when slightly rammed, it will quake. The amount of water is to be decreased during cold or freezing weather. The mixture in general is to be fairly wet. All material should be first thoroughly mixed dry, after which the proper amount of water should be added and the mixing continued until the concrete is uniform. A competent foreman must be in constant attendance at the mixer to give his approval of every batch that leaves the machine.

63. Reinforced Steel.—Reinforcement shall be composed of bars, preferably deformed, of such shape that the bond between the steel and the concrete is not entirely dependent on adhesion. They shall be so combined as to resist fully all stresses in the system known as "tension" and "shear."

Any reinforcement that does not provide for the shearing stresses will not be considered. It is preferable that all shear members be rigidly attached to the main tension members. Sufficient steel must be placed so that the concrete will be obliged to resist direct compression and shear up to 50 pounds per square inch.

No steel shall have at any point less than 1½ inches of concrete covering, except in floor slabs, where ½ inch will be sufficient. The steel for the reinforcement is not to be painted. A slight film of rust upon the steel will not be objectionable, but any bar upon which scales of rust have formed will be rejected.

The elastic limit of the steel shall not exceed 45,000 pounds. The steel shall have an ultimate tensile strength of not less than 60,000 pounds, with an elongation of not less than 20 per cent. in a length of 8 inches. A bar, when cold, must be capable of being bent through an angle of 180° and closed down upon itself without cracking.

All steel reinforcement is to be properly wired or secured together, to prevent displacement while placing and tamping concrete.

Furnish details showing columns, beam connections, splices, stirrups, etc. for the approval of the architects and the Bureau of Building Inspection.

The contractor is to furnish all other ironwork necessary for reinforcing, not especially mentioned above, for the substantial completion of the structural portions of the building.

64. Placing of Concrete.—The concrete shall be deposited in layers not exceeding 6 inches in thickness and shall be compacted as much as possible by tamping. To prevent the formation of bubbles around forms or reinforcement, each layer is to be deposited before the one previously placed has had time to set. When this is impossible, the new layer is not to be joined to the old one until the exposed surface is thoroughly cleaned and covered with a thin grout of neat cement; also, a sufficient amount of steel must extend from old to new work to take up all possible shear. Concrete is to be placed in such manner that the concrete coming next to the forms where the surface is exposed will be composed of fine material.

65. Freezing Weather.—The exposed surfaces of all work deposited in freezing weather must be thoroughly protected. Concrete shall be placed in freezing weather only when it cannot possibly be avoided, and special precaution must be taken to cover the work at once with at least 2 or 3 inches of sawdust, salt hay, or some other approved material. Forms for the work must be left in place at least 3 weeks longer than is customary, and under no circumstances shall they be removed until the concrete has thoroughly set and the frost has entirely disappeared, and then they shall be taken down at the risk of the contractor.

66. Forms.—The centering must be true, rigid, and properly braced; also, it must be of sufficient strength to carry the dead-weight of construction and the force of all tamping without yielding or deflecting. All joints are to be tight so as to prevent leakage of the liquid mass.

The forms must be left in place at least 3 weeks, but in no case are they to be removed until the concrete has attained sufficient strength. The contractor must estimate that the work is to be done with sufficient rapidity, and is to provide enough forms for at least three floors to be in position at one time. Forms shall be removed only at risk of contractor.

For all exposed surfaces, the lumber shall be tongued-and-grooved material planed on the concrete side; also, the face of the form shall be greased with crude oil or soft soap, so that concrete will not adhere to the lumber. After the forms are removed, all small cavities or openings shall be neatly filled and smoothed over with cement mortar.

67. Tests.—All floors shall be tested 40 days after the concrete has been deposited by placing over an entire panel a uniformly distributed load equal to twice the safe live load. With this load there shall not be a deflection exceeding $\frac{1}{800}$ of the span, and the beams or girders shall return to their normal position after the removal of the test load. The expense of such test shall be borne by the contractor. If there is evidence of injury to the concrete work, the architects may order similar tests to be made on other panels. If these show evidence of failure under test, the architects may condemn the entire work and order it to be reconstructed; or, they may cancel the contract, in which case, all that has been paid on the work may be recovered from the contractor. Even if the work is accepted, the contractor must satisfactorily repair any injury that may have been made to the concrete as a result of such test.

68. Allowable Stresses.—The following stresses shall be used in the design of the reinforced-concrete work:

For shearing stresses in concrete, 50 pounds per square inch.

For beams, slabs, and girders, with concrete in direct compression, 500 pounds per square inch.

Tensile strength in steel, 16,000 pounds per square inch.

Shearing stress in steel, 10,000 pounds per square inch.

The tensile strength of concrete shall not be considered in the calculations.

69. Loads.—All floors must carry a live load of 150 pounds per square foot. The roof must carry a load of 30 pounds per square foot in addition to the weight of the structure.

70. Footings.—The concrete footings shall be of the sizes shown in drawings and shall be reinforced with steel bars, the details of reinforcement to be approved by the architects and the Bureau of

Building Inspection before work can be started. Footings along party line are to be so arranged that no footings project over it.

71. Floor Surfacing.—All floors, except the basement floor, shall be finished in wood and the filling is to be of cinder concrete.

72. Basement Floor.—The entire area of the basement floor is to be covered with a 6-inch layer of clean, hard-coal, cinder-concrete filling, mixed in the proportion of 1 part of cement to 4 parts of cinders. After it is thoroughly tamped and graded to the sump pits, lay a $1\frac{1}{2}$-inch coat of top dressing composed of fine-grained grit and cement in the proportion of 1 to 2. The floor is to be laid out in smoothly finished blocks, and is to be so arranged as to drain water to the required pits. Form concrete curbs, sides, and bottom around elevator pit.

73. Gutters on Roof.—The gutters on the roof are to be formed with graded cinder concrete to form fall to openings for conductors marked on the plans.

74. Sleeves, Blocks, Sockets, Etc.—The contractor must build in the concrete work all sleeves, blocks, sockets, etc. required by the owner's electric conduits, sprinkler system, shafting, or other work. He is also to obtain from the architects a set of plans showing the location of all risers or other plumbing pipes, steam pipes, sprinkler system, or any other work that may affect the concrete structure, and is to secure the plans in such time as not to interfere with the progress of the work. The contractor is to leave all scaffolding in place and is to permit other contractors not under his direct control access to same at all times for the execution of their portion of the work.

75. Exposed Surfaces on Exterior of Building.—The exposed surfaces on the exterior of the building where brick does not occur is to be finished in cement, troweled, and left in a smooth condition, free from all tool or form marks.

76. Old Concrete.—Any concrete that has taken an initial set before it was placed or appears to be imperfect in any way is to be removed immediately and replaced without extra charge. Any concrete that has set shall not be broken up and deposited in the work.

COMPLETE AND EXTENDED SPECIFICATIONS FOR REINFORCED-CONCRETE WORK

77. The following specification is about the most complete one used for reinforced-concrete work. It is particularly complete in that it embodies all the requirements of the finished construction. It includes instructions to the contractor as to methods and means of conducting the work, and contains in addition numerous precautionary clauses. A specification of this scope usually places the control of the work in the hands of the architect or the engineer. Some of the clauses contained in the previous specification might be introduced in this specification, as conditions warrant, and these could be changed in detail to suit specific requirements and local conditions.

REINFORCED CONCRETE AND STEEL REINFORCEMENT

78. General.—All the reinforced-concrete work shall be built in accordance with these specifications and the accompanying drawings, to comply with the regulations of the Building Bureau. The contractor is to furnish all labor and materials, including the form work necessary for the complete construction of the reinforced concrete. He is to assume all responsibility for the safety of the work, and is to work in conjunction with the other contractors. He shall provide proper facilities at all times for the inspection by the architects or engineers, and shall furnish, if required, test cubes, 6 inches on a side, made from the regular mixture employed in the work. The cubes shall be delivered to testing laboratories designated by the architects. The contractor shall also supply any pieces of steel reinforcing rods required by the architects or engineers for testing. The cost of the testing of the concrete cubes, steel reinforcement, and cement will be paid for by the owners.

79. Drawings.—Provide and submit for the approval of the architect or engineer complete framing plans showing the size of the reinforced-concrete beams, girders, slabs, columns, and other structural parts. The drawings are also to show the area of the steel reinforcement to be placed in each member and the spacing of the beams and girders.

There must likewise be submitted schedules showing the manner in which it is proposed to bend the steel rods for the several beams and girders, and the spacing of the stirrups and their size must be indicated by diagrams or shown in the schedule. These drawings shall be submitted soon after the closing of the contract, and no form work shall be proceeded with until the drawings have been approved.

80. Allowable Stresses.—The size of the concrete members and the sectional area of the steel reinforcement shown on the drawings shall be closely adhered to, though the unit stresses given in the following shall not be exceeded under the actual dead loads and the proposed live loads, and any beams, girders, or other members, the sizes or reinforcement of which are not marked on the drawings, shall be designed in accordance with these stresses.

The reinforced concrete shall be so designed that the stresses in the concrete and the steel shall not exceed the following limits:

	Pounds Per Square Inch
Extreme fiber stress on concrete in compression ..	500
Shearing stress in concrete....................	50
Concrete in direct compression.................	350
Tensile stress in steel.........................	16,000
Shearing stress in steel........................	10,000

The adhesion of concrete to steel shall be assumed to be not greater than the shearing strength of the concrete.

The ratio of the moduli of elasticity of concrete and steel shall be taken as 1 to 12.

81. Test Loads.—The contractor must be prepared to make load tests in any portion of the reinforced-concrete floor systems within a reasonable time after erection. The test load shall be placed over two entire bays, the location to be selected by the architect or engineer. The tests must show that the construction will sustain a load equal to twice the calculated live load without signs of cracks or excessive deflection.

82. Reinforcing Steel.—The steel reinforcement shall consist of medium steel, with an ultimate tensile strength of from 60,000 to 70,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and an elongation of 22 per cent. Test specimens shall bend 180° around a curve having a diameter equal to the thickness of the piece tested without fracture on the outside of the bent portion.

The steel for the reinforcement must be free from loose scales. Bars covered with scales from rolling, or with heavy rust must be brushed clean with wire brushes. A light rust on the bars will not be considered objectionable.

The steel reinforcement must not be painted nor covered with oil, and if badly covered with mud, clay, or cement, it must be cleaned.

83. Cement.—Only high-grade Portland cements shall be used in the reinforced-concrete construction. Such cements, when tested neat, shall, after 1 day in air, develop a tensile strength of at least 300 pounds per square inch; and after 1 day in air and 6 days in water shall develop a tensile strength of at least 500 pounds per square inch; and after 1 day in air and 27 days in water shall develop a tensile strength of at least 600 pounds per square inch. Other tests, as to fineness, constancy of volume, etc., shall give results in accordance with the standards prescribed by the American Society of Civil Engineers.

84. Sand.—The sand to be used must be clean and free from loam or dirt. The sand may be either bank or river sand, or fine gravel. In any case, there shall not be more than 5 per cent. of loam, clay, earth, or other impurities.

85. Broken Stone.—The stone used in the concrete shall be a clean, broken trap rock, or gravel, of a size that will pass through a $\frac{3}{4}$-inch ring. In case it is desired to use any other material or other kind of stone than that specified, samples of same must first be submitted to, and approved by, the architect or engineer.

86. Composition of Concrete.—The concrete for all girders, beams, slabs, and columns shall be mixed in the proportions of 1 part of cement, 2 parts of sand or fine gravel, and 4 parts of broken stone or coarse gravel. If the broken stone includes the run of the crusher, then the proportions shall be changed so as to make a concrete equally as rich in cement, and to meet the approval of the architect or engineer.

The concrete used in reinforced-concrete-steel construction must be what is usually known as a "wet" mixture. When the concrete is placed under water, it must be placed in a semidry state.

87. Concrete Mixing.—The mixing of the concrete for the principal floor systems shall be done by a batch mixer of an approved type and of a size sufficient for the work to proceed with despatch.

In the smaller portions of the work, where necessity demands it and the approval of the architect or engineer is obtained, the concrete may be mixed by hand as follows:

The cement and fine gravel, or coarse sand, shall be first thoroughly mixed dry, and then made into a mortar by gradually adding the proper amount of water. The crushed stone or gravel shall be spread out to a depth not to exceed 6 inches in a tight box or upon a proper floor, and shall be sprinkled with water as directed. The mortar is then to be evenly spread over the crushed stone, and the whole mass is to be turned over a sufficient number of times to effect the thorough mixing of the ingredients.

88. Placing of Concrete.—Before any concrete is placed, the wooden forms must be cleaned of all shavings or foreign matter, and special attention must be given to the column forms to see that they are cleaned out at the bottom and free from chips, blocks of wood, or shavings.

All the steel reinforcement must be in place and must be inspected and approved by either the owners or architect's superintendent or representative.

It must be observed that the concrete is thoroughly mixed, and the concrete must be deposited in the forms not later than 10 minutes after it leaves the mixer. The work of concreting must be carried along uninterruptedly, unless the floor is of such an extent as to require the work to be stopped off, when it shall be done as stipulated under the heading Stopping Off the Work. In all instances, the slab must be formed in conjunction with the beams and girders. The filling of the beams and girders flush with the top of the slab centering and allowing it to remain will not be permitted. In placing the concrete, it shall be flat-spaded along the sides of all beams and girders and shall be tamped and settled in the column forms by stirring with poles or rods.

In placing the concrete of the slabs, the slab rods, if not arranged with a spacing or registering device, shall be pulled up into the concrete work, or if placed in conjunction with the concrete, they shall be laid upon a thin layer of concrete; the tamping of the forms with iron rods or bars so as to raise the steel reinforcement from the centering will not be permitted. The column forms shall be filled from 6 to 8 hours previous to the beams, and the concrete must be puddled and well stirred. Should the column exceed 12 feet in height, intermediate pockets must be provided in the forms and the form half filled from these. When the intermediate opening is closed, the balance of the form is then to be filled. After the concrete has reached its initial set, the work shall not be jarred by walking or wheeling over it, nor shall heavy planks or form work be dropped upon it.

89. Concreting at Low Temperature.—No concrete shall be placed when the outside temperature is less than 32° F., and no frozen sand, stone, or gravel shall be used in mixing the concrete. If the concrete is placed at or about 32° F., then the water used in mixing shall be heated with a steam jet, or otherwise, and all sand, gravel, or broken stone shall be thawed out, either by exhaust steam or by any other approved method. Salt may be used in the water for mixing the concrete in freezing weather. In no instance, however, shall salt be used for face concrete work. The quantity of salt to be used shall not exceed $1\frac{1}{4}$ pounds for each bag of cement.

After the concrete has been placed in cold weather, the greatest precaution must be exercised to protect it so as to prevent freezing.

This may be done by the methods herein described or by any other approved method. Lay over the top of the fresh concrete a layer of 1-inch rough boards with an air space between the concrete and the boards. Over the boards lay tar paper, or tarpaulins, turning the edges down so as to close the air space between the boards and the concrete; upon the covering so constructed place hay or straw to the thickness of at least 4 inches. Place in the room or story beneath the concrete, if it is a floor construction to be protected, salamanders, preferably using coke.

All window or door openings, wherever possible, shall be covered with muslin to keep out the air.

90. Concreting at High Temperature.—In hot weather the greatest care should be exercised to see that the concrete is properly mixed. The work of mixing shall be carried on under cover, and all broken stone shall be wet down with cold water. Sand, likewise, shall be protected from the heat, to prevent caking or the improper mixing of the concrete.

91. Placing Concrete at Night.—No concrete shall be placed at night unless upon the approval of the architects or engineers, who will see that the inspector in charge is constantly on the premises. If such concreting is carried on, adequate lights must be provided at the mixing machine and throughout the area where the concrete is to be placed.

92. Saving From Jar and Shock.—Care shall be taken when the concrete has been placed and is in the process of setting, that it is not jarred by walking or wheeling over it unnecessarily. It shall not, especially, be subjected to sudden shock by dropping lumber, in the storing of false work, upon it. Where concrete that is setting has to be worked on in order to carry on the operation, boards shall be placed for wheeling or walking over it.

93. Stopping Off the Work.—When the operation of concreting an entire floor system cannot be carried to completion in one operation—that is, where it is necessary to stop off the work—this may be done by either of the following two methods, the architects or engineers, however, reserving the right to determine which method is preferable.

1. The concrete in the beams and girders, as well as that in the slab construction, shall be stopped off along a vertical plane at the middle of the span. If the work is stopped off in this manner, a concrete block, or dam, previously molded, shall be placed under the steel reinforcement of the beams and girders, and the joint above the steel reinforcement of beams, girders, and slabs shall be vertical.

2. The concrete in the beams and girders shall be stopped off directly over the center of columns, and vertical joints shall be arranged so as to allow at least one-half the column for a bearing surface for the future work. The beams and girders shall be filled, and the junction of the slab made directly over the center of all beams and girders, this junction running parallel with the beams and girders. In no instance shall the beams and girders be filled to the top of the slab centering and left in this manner; the stopping of the work must always be arranged so that the slab makes monolithic junction with the beams and girders at least for a distance equal to one-half their width.

94. Cutting of Concrete.—The reinforced-concrete work shall not be cut or drilled for the installation of pipes, electric wiring, or other work without the permission of the architect or his representative. In no instance will it be allowed to cut the concrete away from the bottom of beams or girders or to drill holes in the concrete work at these points. With permission, holes can be drilled in the sides of concrete beams and girders 6 inches above the soffit, and holes may be cut through the concrete floor slab, provided the work is executed in such a manner as not to spall or damage the concrete work seriously.

95. Joining of Old Concrete.—Whenever fresh concrete joins concrete that has set or partly set, the surface of the old concrete shall be roughened, cleaned, and spread with cement mortar, which mortar shall be mixed in proportion of 1 part of cement to 2 parts of sand.

96. Finish of Molded Work.—The concrete molded base course and lintels shall be cast in carefully prepared forms, so that they will present a smooth and true surface. The concrete shall be carefully placed so as to preclude any honeycombed or other defective faces. When the concrete has properly set, all this work is to be carefully patent-hammered, with a six-cut hammer, to a uniform texture and sufficiently thorough to eliminate the plastic appearance of cast work.

97. Type of Steel Reinforcement.—The steel reinforcement for the beams and girders shall consist of square twisted or some other approved form of deformed bars. The columns are to be reinforced with plain round bars, and the slabs with square twisted bars, deformed bars, expanded metal, or some approved form of wire fabric.

In all instances, the steel reinforcement shall be of equivalent area to that shown in the drawings, and where the same is not marked, the work is to be designed to comply with the unit stresses herein given.

98. Method of Forming and Placing Reinforcement. Preference will be given to those types of reinforcement which are made and placed in the forms as a complete fabrication for the beams or girders.

The reinforcement for the beams and girders shall have a due proportion of the rods or bars bent upwards toward the abutments, after the manner of a cambered or truss rod, in order to resist the negative bending moments over and at the bearings. In all cases, the straight portion of such rods at the center of the span shall be equal to one-half the length of the span.

All beam rods shall be sufficiently long at the top, where they bear upon the girders, to lap the full thickness of the girder, and all top rods of the girders shall extend the full thickness of the columns and lap at least 12 inches.

There shall be provided over the juncture of the girders at the column supports two $\frac{3}{4}$-inch bars at least 5 feet long, with ends turned over for securing embedment and bond in the concrete. One of these rods shall be provided over the intersection of all beams and girders.

Stirrups in sufficient number and sizes must be provided in all reinforced-concrete members subjected to tranverse stress. The stirrups shall consist of $\frac{3}{8}$-inch square or round bars, or else of $\frac{1}{8}'' \times 1''$ bars bent in **U** form, and properly wired to the steel reinforcement in such a manner that they will be accurately spaced. Other forms of stirrup construction may be approved. In all instances the stirrups must be capable of providing ample resistance for the horizontal shearing stresses or the oblique tensile stresses.

The stirrups shall be spaced at least 6 inches apart near the abutments, and not farther apart than 4 feet at the center of the span. They shall in all instances be of such a shape and length as to engage with the girder, beam, and slab reinforcement. If the stirrups are of square or round bars, they shall be bent over at the top so as to lap the centering at least 6 inches, and shall be wired to rods extending parallel with the beams or girders to which the slab reinforcement is to be wired.

If stirrups of the flat-bar form are used, they shall be punched at the top, and the slab reinforcement threaded through them or wired to them.

The slab reinforcement shall be so placed as to provide a uniform sectional area of metal, as required by the plans and drawings. If the slab is reinforced with rods or bars, there is to be provided shrinkage rods or bars extending at right angles to the slab reinforcement. These shrinkage bars or rods are to be $\frac{5}{16}$ inch square or round, and are to be spaced at a distance apart of 2 feet from center to center.

Also provide shear rods or bars 5 feet in length, and of the same size as the reinforcing bars, over all girders, extending at right angles to their direction, and spaced at a distance apart of 12 inches from center to center.

If expanded metal is used for the slab reinforcement, care must be taken to see that the long way of the mesh is at right angles to the beams, or in the direction of the slab reinforcement.

If woven or electrically welded wire fabric is used for the slab reinforcement, the cross-wires must be sufficient to insure resistance against shrinkage.

All expanded metal and woven wire must be so arranged that the laps do not occur in the slab spans, but occur over beams and girders. All such laps must be at least 12 inches, and the junctions must be securely wired together. Where electrically welded or woven-wire fabric is used, the reinforcement must extend over several slab spaces; also, it must be brought near the top surface over the beams by propping it up from the slab centering.

The contractor shall provide round bars, of the size shown in the drawings, for the longitudinal reinforcement of the columns. These rods shall be tied with $\frac{1}{4}$-inch wire ties, looped at the ends, so as to fit over the longitudinal rods, and must be spaced not farther than the least side of the column. In no instance shall they be spaced at a greater distance apart than 24 inches. The ends of the longitudinal rods are to be joined by abutting them with pipe sleeves, and four $\frac{3}{4}$-inch lap rods 4 feet long are to be provided at each splice.

Place in all concrete brackets at least two $\frac{5}{8}$-inch round or square bars of sufficient length to interlace with the column and beam, or girder, reinforcement.

Also provide any necessary shear bars at beam, girder, or spandrel bearings where the concrete and the steel reinforcement do not amply provide for this stress.

99. Protection of Steel Reinforcement.—The steel reinforcement of the beams and girders shall in all instances have at least $1\frac{1}{2}$ inches of concrete on the side and 2 inches on the bottom, or soffit, of the beams and girders, and all column reinforcement shall be protected by at least 3 inches of concrete. The slab rods or other slab reinforcement shall have a protection of about 1 inch, and though the showing of a slab rod now and then through the concrete will not be considered sufficient cause for the condemnation of the work, the prevalence of this defect will not be allowed, and every precaution must be taken to prevent it.

100. False Work and Form Construction.—The contractor shall furnish all the necessary material and labor for the proper construction of the false work and forms for molding the reinforced-concrete work. He shall submit for the approval of the architects or engineers sketches or drawings showing the method of the form construction that it is proposed to use. The lumber for the form construction shall consist of dressed sap pine, free from loose knots, wanes, and bad splinters. No hemlock will be allowed in the construction of the form work, although no objection will be made to hemlock braces, studding, and false work.

All form work shall be constructed accurately to the framing plan. The soffit of beams and girders shall be level, and all slab forms shall be true, level, and out of wind. The sides of the forms for the beams and girders shall be plumb, and all work shall be straight, with tight joints.

The box forms of all columns, beams, or girders shall be constructed of 2-inch dressed material. The centering for the floor slabs shall be composed of 1-inch dressed, tongued-and-grooved, matched flooring boards, well battened together with at least three battens to a panel. The slab centering shall be supported upon 2″× 8″ joists spaced from 16 to 24 inches apart. The sides of the forms for the beams and girders are to be held together with 1″× 6″ boards, well nailed, and spaced at the same intervals as the joists supporting the slab centering. The column forms are to be yoked at intervals of 18 inches with slotted scantling or bolts, or with scantling boards, or with scantlings cleated and wedged or nailed at the corners.

The column forms, and all other closed-in form work, where necessary, is to be provided with small handholes for cleaning out the forms. The edges of all beams, girders, and columns are to be chamfered by inserting in the corners a 1½-inch angle mold, and the angle of all slab construction with beams and girders is to be made by chamfering the edge of the forms at an angle of 45° to the thickness of the slab centering.

All centering must be put together with as few nails as possible, using cleats and wedges or other clamping devices in preference to nails or spikes.

All false work is to be constructed of 3″× 4″ studs. If the studs have to be spliced, pieces of 1-inch material equal to the width of the stud are to be placed on all sides. Where numerous studs have to be spliced they must be well braced with cross-bracing and ledger boards.

The contractor must provide a sufficient number of complete sets of forms for the construction to proceed without interruption. There is no objection to using the forms from a finished floor for the construction of an upper tier, provided it does not necessitate the removal of the centering or interfere with the stipulation relating to the removal of forms herein given.

The studding must be placed under all floor construction at centers not farther apart than 4 feet, and all studding must be well cross-braced, especially in the upper stories, to resist wind pressure. All especially heavy construction shall have the supports beneath it doubled, and all trussed work for the support of concrete shall be submitted to the approval of the architect or engineer before being used. If studs or other supports for reinforced-concrete work must rest upon the soil, they shall have placed beneath them, over good soil, a 3″×10″ plank at least 3 feet long. Where the soil is soft, several planks shall be crossed in order to get sufficient support to prevent settling under the weight of the concrete.

The contractor shall observe that the forms are entirely free from all dirt, sawdust, cement drippings, shavings, and other substances. The base of all column forms must be thoroughly cleaned of all shavings and chips through the handhole provided for that purpose, and this shall be done at the time when the first batch of concrete is in position for dumping into the column forms. The forms shall be thoroughly washed down with a hose before the concreting is started.

The greatest care must be exercised in removing the forms and false work. The beam and girder forms shall be so arranged that the sides of the forms can be removed and the bottom form board with its separate studding left in place. When the weather is favorable the forms may be removed inside of 10 days after the concrete has been placed. If the weather is not favorable for the setting of the concrete, as in cool or damp weather, the forms must be left in place for at least 3 weeks. Before the removal of the false work, the architect or his representative shall be notified. He shall, in conjunction with the contractor's superintendent, examine the work, carefully sounding it with a hammer to determine whether it shows a sufficient degree of hardness. In all instances, the false work should be removed gradually; that is, by driving out the wedges from under the centering and allowing it to settle 1 or 2 inches. The work should then be reexamined and the forms entirely removed if the condition of the concrete warrants it. Temporary shores shall be provided under the floor slabs immediately after the removal of the original false work, and these shores shall be left in place for a period of at least 1 week.

Where the concrete has been placed in freezing weathers, special precautions must be exercised in the removal of the form work. The false work must not be removed while the concrete is frozen.

101. Sockets and Fastenings.—Provide, and secure through the bottom form boards with studs or tap bolts, malleable-iron or cast-steel sockets. These sockets must be tapped out full for $\frac{3}{4}$-inch bolts, and must be arranged with cast prongs, bolts, or some other approved form of anchorage in the concrete work. In all instances the anchorage shall be such as to equal in strength the resistance of a $\frac{3}{4}$-inch bolt at the root of the thread. The sockets shall be placed throughout the beams and girders near each abutment, and shall be spaced not farther apart than 5 feet intermediately. They are to be placed in the center of the width of the members and in line with each other at right angles with the beams and girders.

Also provide and place through the center of the span of all beams and girders, close under the slab, four pieces of $1\frac{1}{4}$-inch pipe, spaced

$2\frac{1}{2}$ inches from center to center, for the passage of electric-light wires, gas piping, or other installation.

Also provide any special bolts or other fastenings required by the drawings and properly place them in the position indicated.

102. Openings for Pipes, Etc.— Place where required throughout the work, pipe sleeves. These pipe sleeves shall be placed through the slabs for the installation of the plumbing, steam fitting, or electric wiring. These pipe sleeves will be furnished by the several subcontractors for the equipment, but will be placed by the contractor for the reinforced-concrete work. These sleeves must be properly located and fixed plumb in the forms before the concrete is placed. Provide any boxing or other form work for the installation of machinery or equipment.

103. Apparatus and Equipment.—The contractor shall provide ample equipment, tools, and implements for the successful and rapid completion of the work. He shall furnish the necessary housing for cement, together with bins for broken stone, sand, or gravel, so as to prevent the mixing of these materials. He shall furnish an electrical or steam-driven concrete mixer of approved type and sufficient size. Likewise, he shall provide the necessary power hoists and other apparatus for conveying the concrete, and supply slice bars, tamping tools, and shearing and bending tools for forming the steel reinforcement. All machinery, tools, and implements shall be consistent with the size of the work and efficient for the purpose.

104. Cleaning and Patching.—The contractor for the reinforced-concrete work shall clean down all the exterior and interior face brick, terra cotta, and stonework that may have been damaged or stained by leakage from forms or carelessness in placing. All form work, rubbish, and material shall be removed from the premises upon completion of the work. All concrete that is spalled or honeycombed shall be neatly patched with cement mortar. Care must be taken to see that all portions of splintered forms adhering to the concrete work are removed. The entire work must be left in a finished and completed condition.

COST DATA FOR REINFORCED CONCRETE

COSTS AND METHODS OF ESTIMATING

105. The cost of reinforced-concrete construction depends on the local cost of labor and materials, and the bids that will be received for such construction are considerably influenced by the experience of the contractor bidding on the work.

106. In making estimates upon reinforced-concrete structures, the cost may be closely determined by taking unit prices per cubic yard or per square foot of floor surface from work that has been erected, and using these unit prices for calculating the cost of the new structure. In order that such an estimate shall come close to the actual cost, the plan and specifications, as well as the prevailing prices of labor and materials, must be carefully compared with those of the structure from which the unit prices are derived. Such an estimate, of course, will be only approximate, but can sometimes be made so as to be within 5 or 10 per cent. of the actual cost, one way or the other.

107. In making up an exact estimate of cost, the quantity of materials required can be accurately determined and their cost then found by using the current market prices. As a rule, however, the cost of the labor of mixing and placing the concrete and constructing the form work cannot be so readily determined. Therefore, for these, it is customary to depend on the cost prices that have been found to obtain for work already erected. The cost of the labor for the erection of the forms is particularly difficult to determine, especially where reinforced-concrete construction is new to a locality and the local carpenters have had little or no experience with this class of construction.

The cost of the steel reinforcement may be accurately determined by estimating the number of pounds required and obtaining pound-price bids for the steel. Another way of determining this cost is to submit the plans and specifications to concerns that make a specialty of supplying steel reinforcement. These concerns will quote either for steel delivered f. o. b. or for steel bent, fabricated, and put in place.

In estimating on important work, a good plan, especially where a unit price previously obtained from actually constructed work is used as the basis of the costs, is always to make a detailed estimate and then carefully check it by using the approximate method.

108. Cost per Cubic Yard of Concrete.—The unit prices for approximating the cost of reinforced-concrete construction may be based on either the cubic yard of concrete placed or the square foot of wall or floor surface. They generally include the entire cost of the form work, the steel reinforcement, and the materials and labor necessary for mixing and placing the concrete, as well as the cost of labor to remove the forms and the cost of patching or repairing defects.

Generally, the cost of reinforced-concrete work in floor and wall construction, including the cost of the forms and the steel reinforcement will run from $16 to $20 per cubic yard of concrete placed. The unit cost of reinforced-concrete walls decreases as the wall becomes thicker. For example, building walls from 6 to 8 inches thick, including the forms, cost from $18 to $20 per cubic yard, complete, while thick retaining walls and abutment walls cost from $6 to $7 per cubic yard. Where an earth embankment forms one side of the forms, walls of the latter class can sometimes be built for $5 per cubic yard.

109. Cost per Square Foot of Floor Systems.—The cost of reinforced-concrete floor systems in building construction depends on the clear spans of the beams and girders. The uniformity or lack of uniformity in the sizes of the beams, girders, and columns, as well as the extent of the floor system and the height of the building, also greatly influences the cost. Generally, a reinforced-concrete floor system cannot be constructed for less than 50 cents per square foot of floor surface, and in buildings where the conditions are not favorable for cheap work, the cost is sometimes as much as 60 or 65 cents. The price per square foot of floor surface just mentioned includes the cost of the form work and the metallic reinforcement; in fact, it is the complete price of the reinforced-concrete work exclusive of the finished floor, such as the wood or cement floor finish and the cinder-concrete fill.

110. Cost of Reinforced-Concrete Buildings. For a reinforced-concrete factory building of a good rectangular plan and of the usual story height, namely, 14 feet from floor to floor, the cost of the building per square foot of floor is about $1.50. Where the height from floor to floor is about 15 feet, and the building partakes of the nature of a warehouse, with some pretensions to architectural decorations in the way of cut stonework or terra-cotta trimmings, the cost will be from $1.60 to $1.75 per square foot of floor area. City commercial buildings, with reinforced-concrete floors, windows protected with metallic frames and sash and wire glass, and a front of ornamental terra cotta or cut stone, will cost from $1.80 to $2 per square foot of floor. The cheapest kind of reinforced-concrete building can seldom be built for less than $1.25 per square foot. The cost seldom exceeds $2, however, although it is possible to erect a very ornate building of this type of construction at a cost much greater than $2 per square foot. The prices given include everything but the steam heating and electric-light installation and elevator apparatus.

DATA FOR ESTIMATING QUANTITIES

111. Amount of Steel Work.—Persons engaged in estimating reinforced-concrete work usually have their own short-cut methods of determining the quantities of materials required. In estimating the quantity of steel reinforcement, it is well to use a table that gives the areas and weights of round and square bars. Such a table can be conveniently extended to include the usual deformed bars. Table I gives the areas and weights of the usual round rods and square bars used in reinforced-concrete construction. It will be found convenient in estimating the weight of steel reinforcement.

TABLE I

AREAS AND WEIGHTS OF ROUND-ROD AND SQUARE-BAR REINFORCEMENT

Size Inches	□		○	
	Area Square Inches	Weight per Foot Pounds	Area Square Inches	Weight per Foot Pounds
¼	.063	.212	.049	.167
5/16	.098	.333	.077	.261
⅜	.141	.478	.110	.375
7/16	.191	.651	.150	.511
½	.250	.850	.196	.667
9/16	.316	1.076	.249	.845
⅝	.391	1.328	.307	1.043
11/16	.473	1.608	.371	1.262
¾	.563	1.913	.442	1.502
⅞	.766	2.603	.601	2.044
1	1.000	3.400	.785	2.670
1⅛	1.266	4.303	.994	3.379
1¼	1.563	5.312	1.227	4.173
1⅜	1.891	6.428	1.485	5.049
1½	2.250	7.650	1.767	6.008
1¾	3.063	1.041	2.405	8.178

Generally, it is not necessary to figure the quantity of material required for the stirrups, as this can be taken roughly at 10 per cent. of the total weight of the main reinforcing bars. This 10 per cent. includes such details of light reinforcement as column ties, wiring, etc. In making up the estimate sheet for the steel reinforcement, the weight of each bar of different size should be obtained and tabulated, because the pound price varies with the size of the bar; that is, the pound price for bars smaller than ¾ inch is greater than the base price, increasing in cost as the size of the bars decreases. This variation in price remains uniform; only the base price fluctuates with the market.

112. Estimating Quantities of Concrete.—In taking off the quantities of concrete from plans, it is necessary to determine the number of cubic yards in the columns, slabs, beams, and girders. In calculating such quantities, Tables II, III, and IV will be found useful.

TABLE II

CONCRETE QUANTITIES IN SLABS OF DIFFERENT THICKNESSES

Slab Thickness Inches	Cubic Yards of Concrete per 100 Square Feet	Slab Thickness Inches	Cubic Yards of Concrete per 100 Square Feet
3	.926	5	1.543
3½	1.080	5½	1.698
4	1.235	6	1.852
4½	1.389		

In Table II is given the number of cubic yards of concrete in 100 square feet of slab surface for slabs of different thicknesses. In using this table, it is necessary to determine the floor areas, to divide by 100, and to multiply by the values given in the table under the corresponding thickness of slab. The result will be the number of cubic yards of concrete.

In Table III is given the volume of 1 linear foot of square columns of different sizes. In using this table, it is necessary to obtain only the number of linear feet of each size of concrete column and to multiply by the value in the table to get the number of cubic yards of concrete. If the columns are round, with a diameter equal to that of the sides of a square column, the quantity of concrete required can be determined by multiplying the value given in the table by .7854.

Table IV will be found convenient if it is desired to determine approximately the number of cubic yards of concrete contained in the beams and girders of a floor system. The concrete in the slab is already provided for in Table II, so that in considering the beams and girders, their depth from the under side of the slab is taken. In Table IV, the width of the beam or girder is given in the horizontal row of figures at the top, while the depth from the under side of the slab to the soffit is given in the vertical columns of figures under the horizontal row. The number of cubic yards of concrete required for beams of different sizes and for each 100 linear feet is given in the column at the right-hand side of the table. The values in cubic yards given in the last column are calculated for the minimum beam area contained in the horizontal row to which

TABLE III

CONCRETE QUANTITIES IN COLUMNS OF DIFFERENT SIZES

Size of Column Inches	Number of Cubic Yards of Concrete per Foot	Size of Column Inches	Number of Cubic Yards of Concrete per Foot
6×6	.009	18×18	.083
7×7	.013	19×19	.093
8×8	.016	20×20	.103
9×9	.021	21×21	.113
10×10	.026	22×22	.124
11×11	.031	23×23	.136
12×12	.037	24×24	.148
13×13	.043	25×25	.161
14×14	.050	26×26	.174
15×15	.058	27×27	.188
16×16	.066	28×28	.202
17×17	.074	29×29	.216

TABLE IV

CONCRETE QUANTITIES IN BEAMS AND GIRDERS OF DIFFERENT WIDTHS AND DEPTHS

Width of Beams or Girders, in Inches							Cubic Yards of Concrete for 100 Linear Feet of Beam
12	11	10	9	8	7	6	
Depth of Beams and Girders, in Inches							
						6	.93
						7	1.08
					7	8	1.23
					8	9	1.39
					9	10	1.54
				8		11	1.65
				9	10	12	1.80
				10	11	13	1.98
			9		12	14	2.08
			10	11	13	15	2.26
			11	12	14	16	2.47
		10			15	17	2.57
		11	12	13		18	2.68
			13	14	16	19	2.88
	11	12		15	17	20	3.06
			14	16	18	21	3.24
	12	13	15		19	22	3.34
		14		17	20	23	3.50
12	13		16	18	21	24	3.68
		15	17			25	3.86
13	14			19	22	26	3.91
	15	16	18	20	23	27	4.12
14		17		21	24	28	4.32
	16		19	22	25	29	4.40
15	17	18	20	23	26	30	4.63
16	18	19	21	24	27	32	4.89
17	19	20	23	25	29	34	5.14
18	20	21	24	27	31	36	5.40
19	21	23	25	28	32	38	5.76
20	22	24	27	30	34	40	6.17
21	23	25	28	32	36	42	6.43
22	24	26	29	33	38	44	6.69
23	25	28	30	34	40	46	6.94
24	26	29	32	36	42	48	7.41
25	27	30	33	38	43	50	7.72
26	28	31	35	39	44	52	7.92
27	29	32	36	40	46	54	8.23
28	30	34	37	42	48	56	8.57
29	31	35	39	44	50	58	8.95
30	33	36	40	45	51	60	9.18

they apply. For example, assume that in a floor system there are 900 linear feet of beams 8 inches wide and 16 inches deep. Somewhere in the column marked 8 at the top will be found the number 16, and opposite this number, at the extreme right, the value 3.24 will be found. Therefore, the quantity of concrete in the beams of the floor system is equal to $\frac{900}{100} \times 3.24 = 29.16$ cubic yards.

After the entire quantity of concrete for the job has been determined, the quantity of cement, broken stone, and sand required to make the concrete can be estimated from the table given in *Plain Concrete*. The cost of these materials can then be ascertained, and if the actual cost of mixing and placing is known, the total cost of the concrete work can be determined; or, if the exact cost of the concrete per cubic yard, including the mixing and placing, is known, this unit value may be used with the quantities determined to obtain the total cost of the concrete work.

113. Cost of Mixing Concrete, Etc.—The cost of mixing concrete depends on the quantity to be mixed and the organization and facilities for doing the work. Generally, concrete can be mixed by hand for about 27 cents per cubic yard, and by machine for about 15 cents. Other things to be considered in estimating the cost of concrete work are the hauling of the materials to the mixer, and the wheeling, dumping, hoisting, placing, spreading, and ramming of the concrete. The entire operation of hauling, mixing, and placing, including the foreman's wages, can usually be performed for about $1.30 per cubic yard.

114. Cost of Concrete Ingredients.—Cement is the most costly ingredient used in concrete. Formerly, this material was shipped in barrels lined with paper, but now it is usually sent in cloth or paper bags. Foreign shipments, however, are always made in barrels. A barrel of cement weighs about 375 pounds and contains 4 bags. Its price f. o. b. cars at the mill, exclusive of the cost of the package, is about $1.50 per barrel. If packed in cloth bags, 7½ cents is added for each bag; and if packed in paper bags, 2 cents is added. A rebate of 7½ cents is allowed on all cloth bags returned to the manufacturer.

Broken stone is sold by the ton, which contains about 20 cubic feet. Usually, good broken stone costs from $1.50 to $1.60 per ton f. o. b. cars. This price, however, may vary considerably in different localities.

Sand is also sold by the ton. A ton of sand contains about 1 cubic yard and costs from 50 cents to $1.

The total cost of the materials in a cubic yard of concrete is about $3, so that when the cost of hauling, mixing, and placing is added, the total cost of the concrete in the forms, at the lowest figure, is $4.30.

115. Cost of Steel Reinforcement.—The price of steel reinforcement, as previously stated, fluctuates with the base price of steel and depends on the distance of the operation from the mill or market. Generally, steel reinforcement of the loose-rod type can be bought, hauled, bent and formed, and placed in position for 2¾ to 3 cents per pound. Steel reinforcement of the girder-frame type can be purchased at from 3½ to 4 cents per pound, this price including the placing in the forms.

The cost of placing steel reinforcement in the forms varies considerably. It depends on the weight of the material and whether the reinforcement is made up into girder frames or is in the form of loose bars. Girder-frame reinforcement can generally be placed in the forms for about ⅛ cent per pound, and loose rods can be placed for from ⅙ to ¼ cent per pound.

116. Cost of Form Work.—Form work and centering for the construction of reinforced-concrete floors and buildings will cost from $7 to $9 per cubic yard of concrete placed, or, based on a square foot of floor area, the cost will range from 10 to 15 cents, the former figure pertaining to buildings of simple construction and considerable area and the latter to buildings complicated by brackets, round columns, and other intricate construction.

Column forms will cost about 20 to 25 cents per running foot. The most expensive forms are those used for fine concrete-wall construction. Such forms require considerable time to construct and use large quantities of material; besides, they must be executed with great care if finished work is required. They will cost from 8 to 10 cents per square foot of wall, but will greatly exceed this cost if molded courses, paneled spandrels, and lettering are to be formed.

117. Conclusion.—The preceding figures must necessarily be more or less general because of the wide variation in the cost of materials and labor in different localities. Before making estimates upon the cost of concrete work, the estimator should ascertain the cost of materials and labor in the locality where the work is to be conducted. Likewise, an estimator should always make note of concrete work coming under his observation, and keep cost data and records for future reference. It is only by this means that an accurate estimate can be made of the cost of reinforced-concrete work for a given locality.

FAILURES IN REINFORCED CONCRETE

CAUSES LEADING TO FAILURES

INTRODUCTION

1. In the study of reinforced concrete and its action under the stresses to which it is subjected when used for engineering and architectural structures, a great amount of knowledge about this material will be gained by observing the failures of structures already built and ascertaining the causes of such failures.

Several disastrous failures of reinforced-concrete buildings and engineering structures, such as roofs of reservoirs, stacks, bins, and tanks, have been recorded. Usually, such failures involve not only a serious financial loss, but, owing to the bulk and weight of the material, which, in falling, drags adjoining parts of the building with it, an unusual loss of life, as well. Although failures of structures of any construction are to be deplored, it is evident from what has just been said that failures of reinforced-concrete structures are particularly disastrous. Therefore, in order to impress on the mind the necessity of employing the greatest care and using the best materials in the construction of work in reinforced concrete, it is the intention to show as far as possible the result of failures of various structures and to review the reasons advanced for such failures after close observation by experts in this line.

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One important point to bear in mind is that great opportunities occur for failures and collapse during the progress of construction of a reinforced-concrete building or just at its completion, even though the building would probably possess sufficient strength if it could stand for 2 or 3 months, or until the concrete becomes thoroughly hard.

FAILURES DUE TO DEFECTS

DEFECTIVE CALCULATIONS

2. In the design of a reinforced-concrete structure, as well as any engineering structure, errors are liable to be made in the engineering calculations. About the most common error is that made in calculating the dead and live loads; and often, such a mistake will not be discovered until the completed building displays weaknesses in columns, girders, and beams. Although these defects might not cause actual failure, they would probably cause the architect or owner to reject the building. Thus, it can readily be seen that an error of this kind is liable to prove more disastrous than an error that might be made in figuring the mere strength of an individual beam or in determining the metal reinforcement required. For instance, a mistake in the dead or live loads might involve an error of many thousands of pounds, representing a great percentage of the strength of the structural member, while an error in the calculation of the resistance of a beam would probably involve only a small percentage of the strength of the beam, which would naturally be taken care of by the factor of safety employed.

3. Errors in calculations are commonly caused by making important changes in the structure and neglecting to recalculate the loads coming on the various structural members. If any radical change is made in the design of a reinforced-concrete structure, the drawings should immediately be carefully investigated to determine how much these changes affect the calculations of the live and dead loads and whether or not important columns, beams, and girders are designed with sufficient strength to carry the additional weights imposed by changes in spans or construction. As a rule, there is little possibility of error in usual or normal columns, beams, and girders in a reinforced-concrete building, because these structural members are checked and rechecked to such an extent as to preclude this possibility. However, errors are likely to occur with special beams and girders, especially heavy lintel beams subjected to concentrated loads.

DEFECTIVE DESIGN

4. Ignorance or carelessness in designing is likely to endanger the safety of reinforced-concrete structures. Generally, a defective design is not due so much to ignorance as it is to an effort on the part of the designer to save material. Error is not usually made in an effort to save in the sectional area of the main reinforcing bars, but generally exists because the reinforcing bars do not lap over sufficiently at the beam and girder junctions, or because stirrups or other secondary shear reinforcements are reduced to a minimum or are entirely omitted. A defective design is sometimes the result of adopting a system of steel reinforcement that is not entirely suitable for the structure under consideration. For example, bars with short stirrups are frequently used in very deep beams, so that the stirrups are not long enough to bond into the concrete in the upper part of the beam or slab. Where special types of reinforcement are used, their details of construction should be carefully investigated in order to avoid defective design. Weaknesses should be corrected by the introduction of secondary reinforcement, such as extensions to the stirrups, lap or tie-rods, etc. Investigations of failures due to defective design have nearly always shown that the beams or girders failed near the columns or abutments, usually in an oblique direction, and that the failure was due in nearly every case to insufficient stirrup or secondary reinforcement. There is probably no record of a serious failure that was due directly to insufficient area in the main reinforcing bars. Even in instances where rods have been known to be omitted from beams, no bad effects have been experienced, because, usually, the omission of one or two rods in a beam caused only a comparatively small reduction in the total sectional area of the steel reinforcement.

DEFECTIVE MATERIALS

5. Failures due to defective materials, steel especially, are very rare. The manufacture of steel has become so standardized that it is hardly possible for a manufacturer to deliver a grade of reinforcing bars or rods so poor in quality as to endanger a structure. The greatest danger to the safety of a reinforced-concrete structure exists in the possibility of using defective cement. The cement industry is not upon such a standard basis as the steel industry, and new companies are constantly entering the field. The usual trouble with cement is probably due to the fact that mills furnish what is called *green cement;* that is, cement that is not allowed to age in the stock bins, but is shipped as soon as it is manufactured. Such cement may prove unsound in setting. Again, it frequently happens that the cement has been improperly proportioned, or contains too great a percentage of gypsum. Such cement, while it may set rapidly, will in time deteriorate and lose most of its strength. Although no actual failures can be traced directly to the use of poor cement, it has been necessary in many instances to take down entire structures, involving a great loss. For important operations, cement is usually tested to determine its quality, and failure because of defective material is therefore remote.

Defects of a serious nature may be developed in a finished reinforced-concrete structure by thawing out frozen concrete after the forms have been removed. Sometimes this frozen concrete, after thawing, will set and become comparatively strong; at other times it will not obtain strength or hardness sufficient to resist the stresses to which it is subjected, and consequently must be taken down and rebuilt.

FAILURES DUE TO CARELESSNESS

POOR WORKMANSHIP

6. Poor and defective workmanship is a great source of inferior concrete construction. In fact, no other cause is likely to produce more serious consequences, with the chance of collapse and the attending loss of life and property. Careless workmanship may be displayed in the erection of the centering, in the placing of the steel reinforcement, in the mixing of the concrete and its placing, and in the care of the work after the concrete has been poured.

7. One of the most serious reinforced-concrete collapses was traced directly to a combination of the just cited causes. The verdict of the coroner, in speaking of the superintendent in charge of the construction, is as follows:

"He allowed the cement to be put in the forms without proper tamping, so that holes were left in it, and allowed four steel rods to be put in a column where the plans called for six rods, and allowed four rods to be put in column 47 when the plans called for eight rods, and by gross and culpable negligence permitted leaves, sawdust, chips, and blocks of wood to be put in the column, thereby endangering the safety of the building."

The manager for the owner was likewise held accountable by the coroner, because by his neglect the defective workmanship was permitted. Quoting from the verdict, the coroner says:

"He had the power to change the working plans and have removed defective construction and have workmen discharged and even to stop the work. He, by gross and culpable negligence, allowed the contract to be violated so as to endanger life by permitting column 47 to be constructed far weaker than was called for by the original plans made under his direction, and weaker than allowed by the building ordinances of the city, although the contract specified that the plans should conform to such ordinances; and he was guilty of gross and culpable negligence when, recognizing his responsibility of inspection of the work in progress, he allowed to be employed a teamster, utterly devoid of experience in building, as sole inspector, without instructions as to the importance of the work he was inspecting, so that the moral effect of such inspection was bad, except possibly for impelling haste in the work rather than care for its integrity. He was guilty of gross and culpable negligence in not removing work which was obviously defective, like blocks of wood and joints of sawdust at the bases of columns, when the contract specified as follows: 'The contractor agrees that the owner shall have power to cause any improper, inferior, or unsafe work to be taken down and altered, and that all improper, inferior, and unsafe work will be taken down and altered at the cost of the contractor, and without charge or expense to the owner * * * and if at any time the contractor shall refuse to remove any work or materials rejected by the owner, then and in that case, the owner, or such other agent as the owner may designate, may, upon five days' notice, declare this contract forfeited * * * the same as if the contract had never been made.' "

8. Poor Forms.—Defective workmanship upon the forms means not only unsafe work, but unsightly work, as well. In reinforced-concrete buildings, girders and beams are frequently badly twisted and sagged, because the forms are so poorly constructed that they cannot properly bear the weight of the concrete. While such carelessness may not produce an unsafe building, it is usually a criterion of the remainder of the work.

9. Misplacing of Steel.—The greatest element of danger exists in the misplacing of the steel reinforcement, in leaving it out of the forms altogether, or in placing it so carelessly as not to have it near the position upon which the calculations were based. Workmen on reinforced-concrete structures seldom understand the principles underlying the construction, and for this reason they are liable to place the steel reinforcing rods for slabs near the top instead of near the bottom surface. In a like manner, they frequently distributed the steel anywhere throughout the mass of a girder or beam, irrespective of the fact that it is supposed to be placed close to the soffit or under side. Carelessness also frequently exists in not properly lapping the steel reinforcement at junctions.

10. Dirty Concrete.—Weakness in reinforced concrete is often caused by permitting foreign substances to remain in the forms. Upon inspection, reinforced-concrete columns are frequently found to be materially weakened because embedded in them are shavings, chips of wood, blocks, and other building materials.

11. Badly Placed Concrete.—A defect that cannot be attributed entirely to careless workmanship, but rather to ignorance or haste on the part of the foreman, is that caused by placing the concrete of girder and slab constructions at different times; that is, in pouring the concrete of a beam or girder and allowing it to set before the concrete of the slab construction is placed. As the slab is often designed to act in conjunction with the beam in resisting the bending stresses, placing the concrete in such a manner will cause an element of weakness and possible danger.

PREMATURE REMOVAL OF CENTERING

12. Removing the centering before the concrete has had time to set properly is the cause of the greatest number of reinforced-concrete failures. The removal of the centering is frequently hurried because of the desire to use the centering material on another floor, or because the floor space is wanted by the other mechanics in finishing the building. Failures due to this cause usually show that the concrete has not set in the usual time, probably because a very slow-setting cement is used or because the concrete is placed in freezing weather. The most serious consequences are attendant upon the removal of the centering or form work before it has been ascertained whether or not the concrete has set sufficiently to attain strength enough to support its own weight and the weight of material that has been placed upon it during construction. In many instances, centering has been removed prematurely because of misunderstood orders and in other instances it has been weakened by the removal of certain portions of the forms in order to facilitate the setting of the concrete by allowing the sides of beams and girders to become exposed to the air.

13. The partial removal of the centering, or supports, was the cause of a serious collapse that occurred in the city of Philadelphia. The building in question, known as the Bridgeman Building, collapsed to such an extent as to destroy nearly one-half of the structure. The following quotation, from an account of the investigation of this failure, illustrates how serious a partial removal of the forms, due to a misunderstood order, may be to a reinforced-concrete operation:

"At the time of the accident, the first three floors had been completed and the forms removed, the newest concrete in these lower floors being about a month old. All the columns supporting the roof were finished and on the north half of the building, where collapse occurred, the roof girders and slabs had been in place 5½ days. Form work was going on at the time upon the griders and slabs of the southern slope of the roof, and the girder spans of 17 feet were supported by seven struts spaced about 2 feet from center to center. According to the regulations of the city of Philadelphia, these struts should have been allowed to remain in place 2 weeks from the time of depositing the concrete. However, on the day of the collapse the subforeman in charge of concreting ordered a negro laborer to remove every other strut from under each of the roof girders, thus leaving the girder to be supported by three or four struts, according as the laborer understood the order. The subforeman did not remain to see this work started, but left for another job which his company was building. The negro transmitted the order in turn to some Italian laborers, who, from ignorance of the language, misunderstood, and proceeded to knock out every strut under girders holding concrete which had been deposited about 5½ days previous. It is not known just how far this action proceeded, but beginning at the back of the building, where the removal was probably started, the whole existing roof structure gave way, and pulled with it the girders and columns, breaking through each floor in succession. The most northerly bay of the first floor was broken and the whole mass of the débris carried through to the basement. The bay directly south of this, on the first floor, held under the heavy impact of the falling mass, and although the girders and beams were badly cracked, the floor system did not give way."

FAILURES BEYOND CONTROL

14. Failure Due to Fire.—Probably the only serious failure or collapse of a reinforced-concrete structure that could occur from fire would be caused by the burning away of the supports, or form work, during the construction of the building. A review of the several serious fires that have occurred in reinforced-concrete buildings shows that, while the concrete has frequently been spalled and broken off from the under side of the beams, in many instances leaving the steel reinforcement exposed, the damage has been limited to this extent, and that there has been enough reserve strength in the beams, girders, or structural members to support the load of the floor.

15. Failure Due to Earthquake.—The only test by earthquake that reinforced-concrete buildings have been subjected to occurred during the San Francisco disaster. Although the buildings of this material at that time erected were few, and although a comparison with steel or stone buildings could not be readily made on account of the exposures and conditions being different, it was generally demonstrated that the reinforced-concrete buildings withstood the effects of fire and earthquake better than did the fireproofed steel-frame structures. However, several buildings of concrete partly reinforced—that is, with a very small percentage of reinforcement were seriously injured.

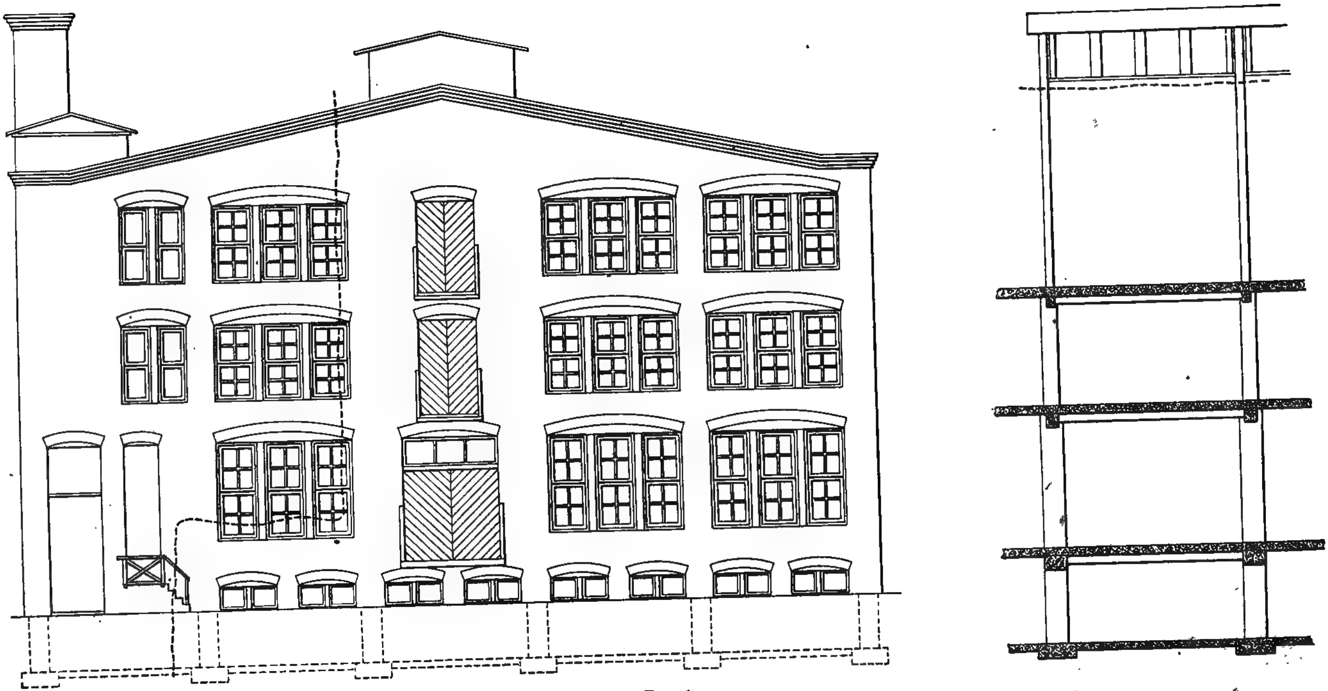

Fig. 1

CONCRETE FAILURES

REVIEW OF PRINCIPAL DISASTERS

FAILURES OF MANUFACTURING BUILDINGS

16. The failure, on July 9, 1907, of the reinforced-concrete factory building that was being erected in Philadelphia for the Bridgeman Brothers Company is one of the most serious on record. In this disaster, two of the workmen were killed and thirty others were seriously hurt. The building was of reinforced-concrete, skeleton-frame construction and three stories in height, exclusive of the basement. A section and elevation of the building is illustrated in Fig. 1, the portion that collapsed being at the left of the broken line. A detail of the construction at the girder and column junctions is shown in Fig. 2. In this failure, nearly one-half of the building above the first floor collapsed and fell into the basement. The floors were designed for live loads of 180 pounds on the first floor and 120 pounds on the second and third floors. The reinforcement of the beams and girders consisted of round, straight rods, and the beams and girders were provided with shear stirrups formed of round bars bent in the shape of a stirrup, or **U**. It seems that these stirrups were not definitely located upon any of the drawings, and an examination of the failure revealed the fact that in many instances they were apparently omitted. The floor slabs were reinforced with 3-inch wire mesh. Although there were some features in the design of the reinforced-concrete work

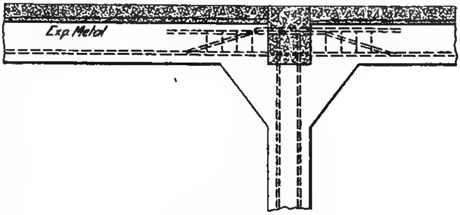

FIG. 2

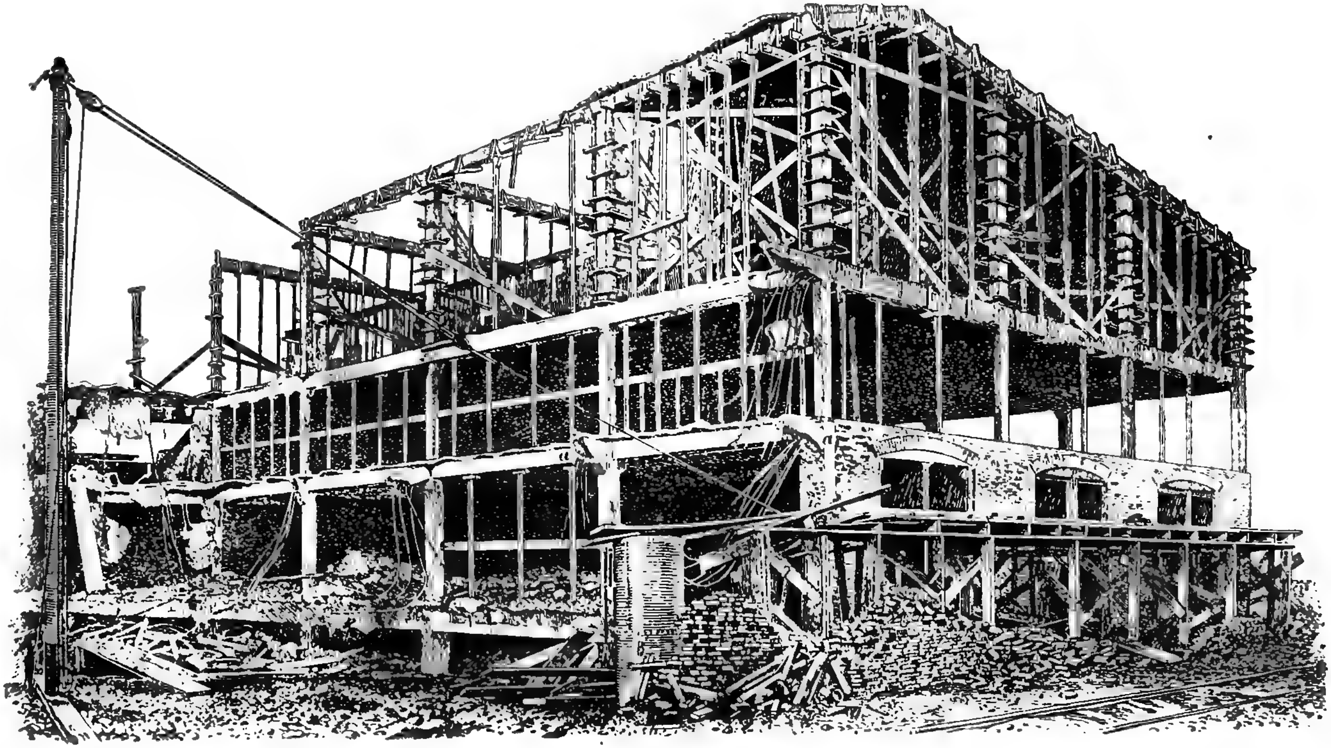

FIG. 3

for this building that might have been improved upon, it is conclusive that the collapse was caused by the premature removal of the centering supports, and if this had not occurred, the building would undoubtedly have been substantial and sufficiently strong for the purposes for which it was designed and erected. Fig. 3 shows the appearance of the collapsed portion of the building, and an examination of this figure will reveal a typical reinforced-concrete failure at the junction of the girders with the columns. In nearly all failures, beams or girders break diagonally, as shown at *a*, particularly where the stirrups have been left out or have not been provided for. The figure also illustrates the completeness of the collapse, and shows clearly how the concrete, which had not properly set, had stripped from the reinforcing rods. One of the surprising features of this failure is that certain portions of the first floor over the basement sustained the weight of the wreckage of the three upper stories. The girders here, however, developed shear cracks.

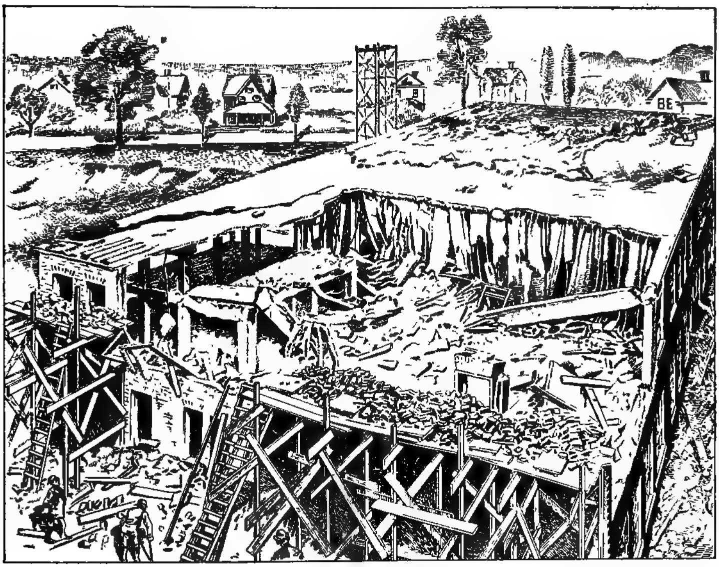

FIG. 4

17. On November 21, 1906, a failure equally as serious as that just described, but considerably more extensive in area, occurred in a portion of a building that was being erected at the works of the Eastman Kodak Company, in Rochester, New York. The collapse of the building, which is shown in Fig. 4, killed two men outright and hurt several others. In this building, a system of hollow-tile and reinforced-concrete joist construction was employed, and the reinforcing metal consisted of Kahn bars, together with plain, round rods. At the time of the accident, the construction of the building had proceeded to such an extent that the exterior brick walls were completed and the roofing material was being put in place. The roof, for a space of about 53 ft. × 89 ft., and its supporting columns fell and beneath the roof a mezzanine floor gave way over an area 21 ft. × 45 ft. The main floor of the building withstood the shock of the falling mezzanine floor and roof, with the exception of a hole about 2 feet square, which was punctured by the end of a falling girder. The cause of this failure was not due in any way to poor design, but was rather caused by careless workmanship and poor material. Following is part of a report made to the coroner:

"Many of the posts show a horizontal joint of sawdust, shavings, and paper. In column 2, at the level of the mezzanine floor, I thrust in my knife blade 2 inches deep. In column 37, at the level of the mezzanine floor, there is a joint of sawdust and shavings, horizontal, nearly if not quite through the section, into which a knife blade was thrust $2\frac{1}{2}$ inches, easily.

"In the north face of column 37 there is embedded a piece of wood showing $5\frac{3}{4}$ inches wide by about 13 inches long, and probably 2 inches or 3 inches thick, set up at an angle of about 45° so as to reach clear across the face of the post, apparently dropped into the wooden form along with the shavings and sawdust before the concrete was put in above the mezzanine floor. In the same column below the mezzanine floor the concrete is so poor that I dug into it with a small stick nearly 2 inches in one spot.

"In many places the steel bars show at the surface of the girders on the side.

"The girder from column 2 to column 3, although in the face of the wall, bulges out so as to be deformed, apparently due to carelessness in bracing the forms.

"The fact that the main floor resisted without appreciable injury the great shock of falling girders, posts, and roof indicates that in general the design is satisfactory. The only place broken in the main floor is where the roof girder between columns 49 and 24, which spanned 28 feet and sustained 560 square feet of roof, slipped off from its bearing on 49, while still being held up by 24, and dropped one end 14 feet and still stands in its leaning position, although of course the attached roof was shaken off.

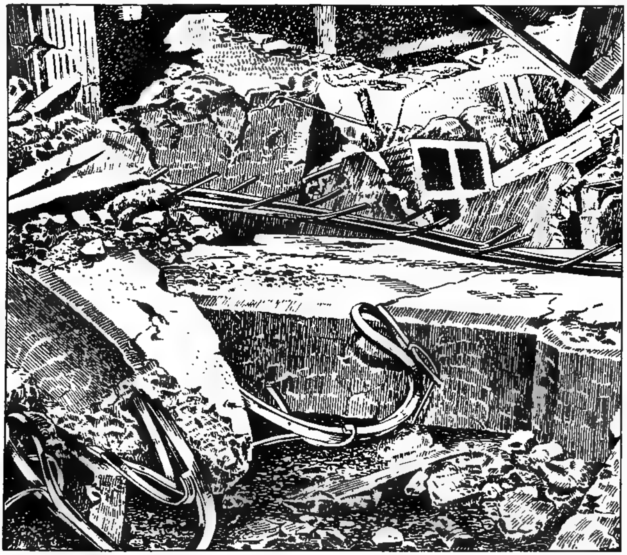

Fig. 5

"The general condition of the building indicates that the cement was good and that ample time was given for setting before the removal of the woodwork.

"In column 47, the upper portion, about 7 feet long, was of such poor concrete that we easily picked off with the fingers the upper 2 feet, and could crumble most of it in the fingers. I afterwards concluded that this concrete was disintegrated by the wrenching off of the roof girder. The lower portion of the column was pretty well covered up with débris, but appeared to have settled down in its own location by breaking. We afterwards dug down and exposed this.

"I saw many places in posts and girders where the concrete was not filled with mortar and had the open texture of a pop-corn ball, indicating that it was put into the molds in too wet a condition.

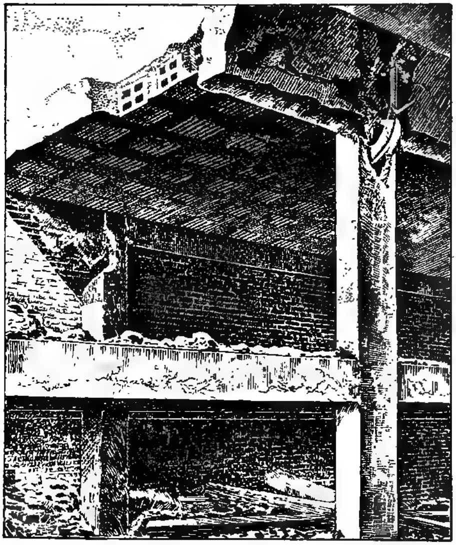

Fig. 6

"The cinder concrete on the roof has the characteristics of earth rather than concrete.

"In many cases the steel bars in girders do not connect or overlap over posts.

"The fallen posts show that there was no connection of the steel bars in the part of a post above a floor and below, and there were no dowels.

"The condition of the wreckage indicates almost (if not quite) with certainty that column 47 failed first, and below the mezzanine floor."

Other details of this accident are shown in Figs. 5, 6, and 7.

FAILURE OF A CHIMNEY

18. There are few instances on record of the failure of reinforced-concrete chimneys. One of the latest to be recorded occurred on January 19, 1907 This chimney, which was located at Louisville, Kentucky, failed by the collapsing of the entire upper section. As the chimney had been erected for about 2 years, the concrete had undoubtedly reached its proper set. The wind storm that caused the accident, although it lasted all day, reached only the maximum velocity of about 42 miles an hour. That this storm should destroy the stack seems strange, because it was known that a wind velocity of 54 miles an hour had been reached in the preceding May. The direction of the wind was from the southwest, and on a side of the chimney that was entirely exposed; the chimney was protected on the northeast side by a ten-story building. It was evident that some fear had been felt for the safety of the chimney, because an attempt was made to run guy lines; these were not attached, however, as the workmen were driven down on the morning of the accident by the severity of the wind.

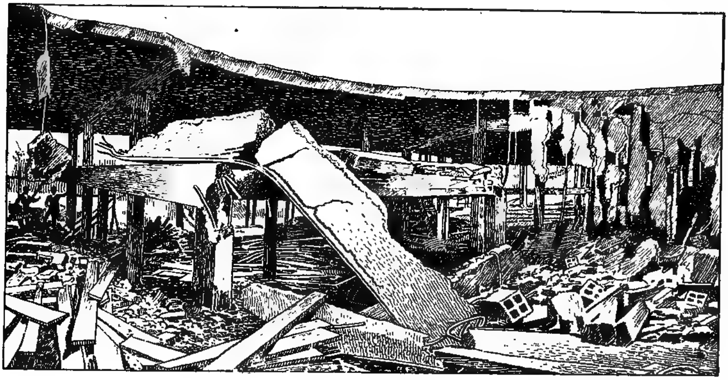

FIG. 7

The chimney was about 176 feet in height above the ground, with a projection above the boiler-house roof of 130 feet. It was, as is customary with reinforced-concrete stacks, made in two parts, the lower part being larger in diameter because it was provided with an inner shell. The entire upper part of the chimney above the offset collapsed completely. The lower 70 feet, which remained intact, was 84 inches in diameter outside and consisted of two concrete shells 5 inches in thickness with a 5-inch air space. The upper portion was 64 inches in diameter outside and consisted of a single shell 5 inches thick.

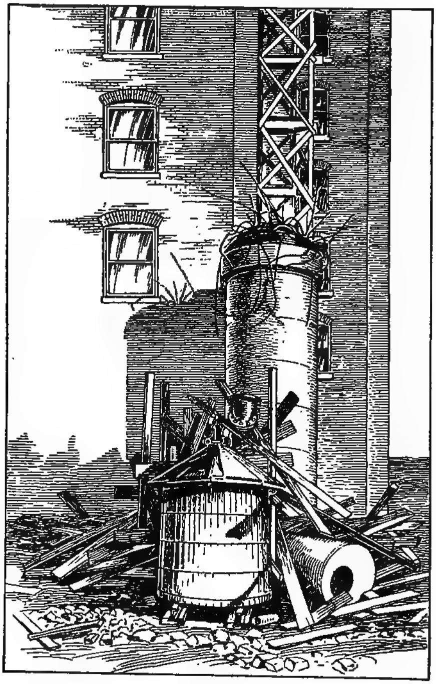

Fig. 8

Owing to the complete wreckage of the upper part of the chimney, the exact amount of reinforcement used could not be determined, but from a careful inspection it was concluded that there were about sixteen $1\frac{1}{4}'' \times 1\frac{1}{4}'' \times \frac{3}{16}''$ **T** irons arranged in pairs for the vertical reinforcement. The circumferential reinforcement consisted of $1'' \times 1'' \times \frac{1}{8}''$ tees spaced about 2 feet from center to center and bent to form hoops. As nearly as could be observed, the circumferential reinforcement was not attached in any way to the vertical.

19. From the evidence of the collapse, as shown in Fig. 8, the chimney had failed directly at the junction of the upper portion with the lower, and had fallen in its entirety, crushing flat when it struck the ground. The portion of the chimney remaining displayed vertical and horizontal cracks not shown in the illustration.

The composition of the concrete was in the proportion of about 1 part of cement to 8 parts of sand, and that its quality was decidedly defective is evident from the fact that portions of it were so soft that they could be disintegrated by rubbing them between the fingers, while many pieces could readily be broken with the hand. Usually, in reinforced-concrete chimneys, the interior shell is subjected to some deterioration by dehydration. Although, in this instance, the interior of the shell was somewhat discolored, the concrete did not seem to be any poorer than that in the other parts of the structure.

A survey of the ruins showed that the concrete was completely stripped from the steel and that there was hardly a piece in the entire mass adhering to the reinforcement. The steel was entirely free from rust scales and was not broken in many instances. The reinforcing bars in the lower part of the stack projected above the line of fracture about 9 feet, as shown in the figure, and were of uniform length. These bars were not broken, but had evidently pulled from the concrete.

In making calculations regarding the moment on the stack, it was determined that with a wind pressure of 50 miles per hour the concrete was not overstressed, at least not for good material, the calculated unit compression being about 480 pounds. As the steel was not broken, the evidence would indicate that the chimney in falling pulled the reinforcing bars from the concrete on the windward side, these bars having been loosened by previous wind storms, and by the repeated gusts that occurred that day. The collapse of this chimney can probably be traced directly to the use of a rather lean concrete and to the fact that there were used reinforcing bars that had no deformation and, consequently, no strong bond.

FAILURE OF BUILDING UNDER TEST LOADS

20. One of the first reinforced-concrete failures on record occurred in 1903 while placing the test load upon the floor construction of a building erected for the J. L. Mott Iron

Works, at Trenton, New Jersey. At this time reinforced-concrete construction was comparatively new, especially in the United States, and it is doubtful whether such an accident as a complete collapse under a test load has occurred since. The building in question was about 300 feet long and 50 feet wide, with four reinforced-concrete floors, a reinforced-concrete roof, and brick side walls. The construction was supported by a center row of reinforced-concrete columns and girders. The second floor was so loaded as to throw most of the weight upon the girders, and here the test load amounted to 225 tons, spread over an area 30 feet square. This test was successful, but on testing the third floor, a test load of 150 tons was placed on one day and allowed to remain over night. On the next day, the balance of the test load was placed until all but ten bags were loaded on the floor. The concrete at this point in the testing began to crack and a portion of the floor broke, falling on the second floor, which was carried with it to the first. The floors were designed for a live load of 250 pounds per square foot, and the specifications stipulated that the test load should be twice as great as the live load.

21. An examination made after the collapse showed that the reinforcing rods consisted of plain, round steel bars, some of which were bent in truss form, and that both the beams and the girders were provided with stirrups. It seems that the portion of the building that failed consisted of the secondary beams, which had a span of 23 feet, and that these apparently failed near the wall end, probably by shearing. A considerable portion of the reinforced-concrete floor remained suspended in the air by the steel reinforcing bars of the other part of the construction. It is evident that the beams must have failed close to the wall, because most of the bags of sand used to make up the test load were found deposited in the cellar along the side walls. The walls were slightly buckled, but the columns and footings showed no appreciable settlement or injury. The conclusion to be drawn from this collapse is that the shears at the abutments were not sufficiently provided for by ample stirrups, or that the beams did not have sufficient bearing on the walls and were not well anchored, or bonded, into them, or that the reinforcing bars or rods did not have sufficient projection on the bearing.

FAILURE OF A BATH HOUSE

22. The failure about to be described, while insignificant in its extent, as it involved only a one-story building, contains several lessons that illustrate clearly the disastrous results of carelessness, poor design, and poor workmanship when combined in the construction of a reinforced-concrete building. All the fault of the collapse cannot be blamed entirely on the reinforced-concrete structure, as the bearing walls on each side of the building, as well as the method by which bearing was obtained for the concrete work, were totally inadequate. That the collapse was complete and extended over a large part of the structure is clearly shown in Fig. 9. Like all reinforced-concrete failures, the mass of the concrete was pretty well broken up, the concrete being almost entirely separated from the reinforcing bars and rods.

23. An investigation of this wreck showed that there were several causes, or a combination of conditions, that made the collapse possible. An examination of the concrete showed that it was badly affected by the frost, which had penetrated from the surfaces of the girders to varying depths. That it was badly frozen was evinced by the fact that some of the concrete was crumbled, and large portions of it could be broken off by a slight effort of the hand. As it is customary in the design of reinforced-concrete constructions to include a portion of the slab in the calculations for the strength of the beams and girders, the girder and slab materials must be placed at the same time, making a monolithic **T** section; or else there must be provided ample stirrups that project from the beam into the slab and interlace with the slab reinforcement. A careful examination of the wreck proved that the material in the girders was different from that in the slab.

The former was composed of trap rock, while the latter was constructed with limestone. Consequently, the girders, which had a span of 22 feet, did not get the benefit of the **T** section formed by the slab, as they were not placed at the same time. The effective depth of the girders was therefore much reduced. This faulty construction, coupled with the frozen concrete, which should never have been left standing without centering, was undoubtedly the cause of the failure.

Again, the investigation showed that the reinforcement did not extend into the bearing wall, because the beam broke

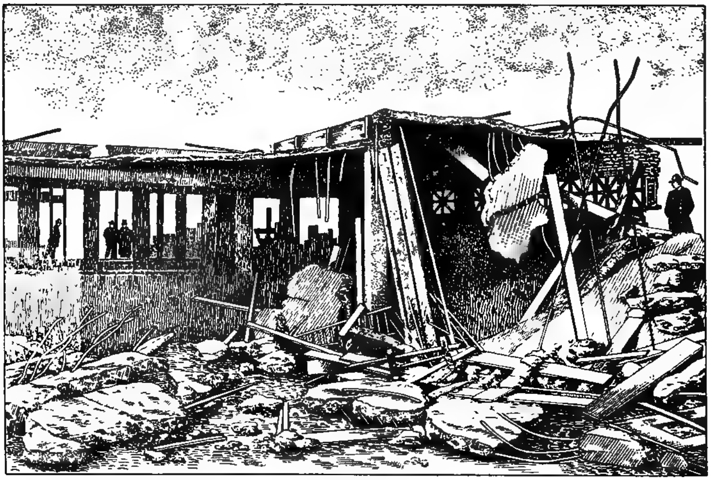

Fig. 9

away from the walls by direct shear. This would not have happened if the reinforcing bars had had ample bearing on the wall. Not only did these faults exist, but some of the bearings of the beams were defective. For instance, an investigation of Fig. 9 shows that the reinforced-concrete construction was supported upon a latticed girder, not directly, but upon the top of an 8-inch wall resting upon this girder—an inadequate support for such heavy construction as reinforced-concrete work. Another defect was observed in the construction of the pilasters that were supposed to act as a bearing for some of the beams or girders. These pilasters, which were 4 inches deep and 2 feet wide, were constructed with the brick wall, but did not bond into it at all, as shown in Fig. 10. Here, the entire brick pilaster had no bond with the wall, but was merely held in place by the adhesion of the mortar. Thus, a portion of the building was practically supported on piers only 4 inches thick and 2 feet wide, having very little strength if predisposed to buckle in a direction toward the interior of the building. Coupled with these defects, the mortar used for the construction of the brickwork exhibited marked defects, being extremely lumpy and evidently devoid of the proper quantity of cement.

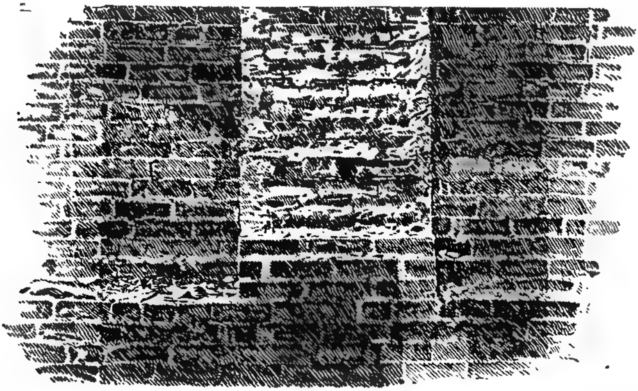

Fig. 10

This collapse is an example of the damage that may be done by the failure of a comparatively unimportant building, so far as the cost is involved. Such buildings, when designed in reinforced concrete, should receive the same care as more important structures, as their collapse is apt to cause loss of life.

FAILURE OF A STORE BUILDING

24. It frequently happens that even where the greatest care is exercised by the designers to provide for the security of good work and its protection, trouble may be caused by poor workmanship. The following instance illustrates that no matter what safeguards the engineer or architect may employ in the way of specification clauses, it is only by rigid supervision that danger of collapse is avoided during the erection of reinforced-concrete buildings. On December 15, 1904, there occurred a collapse of a three-story department-store building of reïnforced-concrete construction, the ruins of which are illustrated in Fig. 11. This accident, fortunately, was attended with no loss of life. Both the floors and the walls of the building were of concrete, the floors being reinforced with wire-meshed fabric.

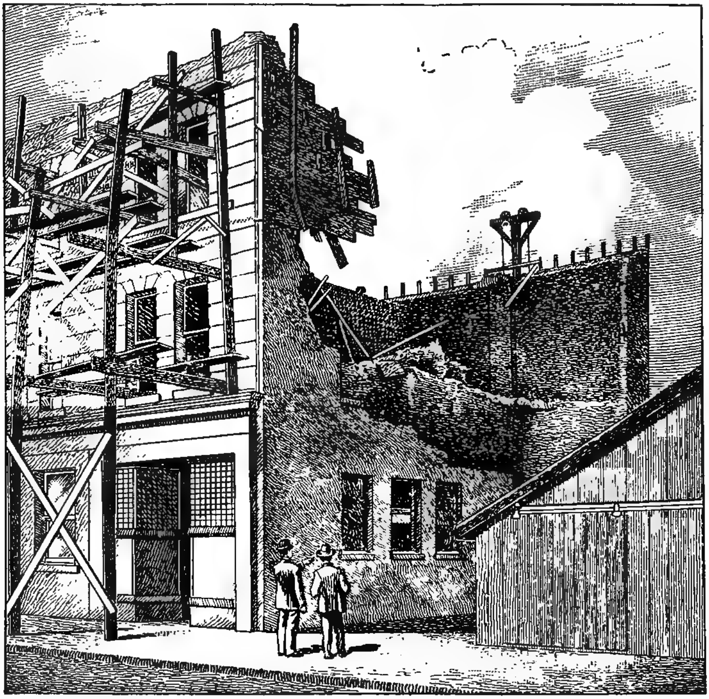

Fig. 11

The architects, appreciating the danger of defective material and workmanship in this class of construction, stipulated that the work was to be accepted only after tests made by the contractor in the presence of the architects. The tests were to be made on sections 5 feet square, loaded to 400 pounds per square foot on the first floor, and 200 pounds per square foot on the second and third floors, at least two sections on each floor to be tested. While the first-floor concrete withstood the test and was found to be satisfactory, the architects, on investigation, believed that the second and third floors would not, and they so notified the contractor's bondsmen. When these two floors were tested, the work showed unusual deflection, and serious cracks and other signs of weakness developed.

The architects' statement is as follows: "We deem the failure entirely attributable to the fact that the reinforcement of the concrete in that part which failed was almost entirely omitted." Concerning the quality of the parts that had been tested, the architects say: "They showed very excellent results, giving exceedingly small deflections under test load," and further state, "We would only emphasize the fact that the metal reinforcement has to be properly put in to secure stability and the strength required as described in the tests." An examination of the collapse also revealed that the concrete itself might have been better.

FAILURE OF A HOTEL BUILDING

25. An important failure of a reinforced-concrete hotel structure located at Long Beach, California, occurred on November 9, 1906. This building, known as the Bixby Hotel, was a typical example of a reinforced-concrete failure, and, as is so often the case, several causes seemed to lead to this failure. The best report of this failure is that made by John B. Leonard, civil engineer, of San Francisco. This report is as follows:

"The girders of the first floor are 14 inches wide by 24 inches deep, reinforced with three bars having a sectional area of 3.62 square inches. On the other floors, the girders are 12 inches wide by 18 inches deep, and the superintendent insists that they have the same reinforcement as on the first floor. The girders are constructed **T** shape, and have .78 square inch of metal for continuity reinforcement, which projects about 3 feet 6 inches each side of column center. None of the girders is provided with haunches or brackets.

"The slab construction consists of 5″×8″ concrete beams, placed 17 inches center to center, and reinforced alternately with bars having a sectional area of .38 square inch and .78 square inch, respectively. Between the beams is a row of 6″×12″ hollow tile, which is placed in a line normal to the girders and parallel to the beams, the 12-inch dimension of the tile being horizontal. On the top of the tile was placed a 2-inch layer of concrete, which contained no reinforcement.

"The columns are built of concrete and contain, usually, for reinforcement four ½-inch round bars. In the wrecked portion, the basement columns are 21 inches square, while those of the first story are approximately round, being about 26 inches at the base and tapering to 22 inches at the top. Their reinforcement consists of four ¼-inch square bars. The second-story columns are 12 in. × 12 in., and the third-, fourth-, and fifth-story columns are 10 in. × 10 in. In some cases there was evidence of wire wrapping about $\frac{1}{16}$ inch diameter, spaced about 16 inches; in other cases, there was no evidence of wrapping whatever. In several instances, some of the bars were as far as 6 inches from the edge of the column and in others only ¾ inch. The rods in the columns at the floor levels are made to abut one another by means of a gas-pipe sleeve. The ruins give evidence that in some instances even this precaution was omitted. In no case was there any evidence that reinforcement had been provided to enable the columns to transmit transverse moment to the girders and columns below. There was ample evidence that the work had been constructed by casting the column up to the elevation of the bottom of the main girders. A period of time had been allowed to elapse before the girders were fabricated. After a second interval of time, the construction of the column for the next story was commenced. This was apparent because of the smooth fractures which occurred at these places. Such reinforcement as was provided in the columns was insufficient to overcome the weakness of the lower joint, and there was none at all at the floor level to safeguard the possibility of a weakness at this point.

"In the wall columns, there was an eccentric condition of loading through the imposing of a 16-inch column on a 21-inch column, whose outside faces were flush.

"The wall construction consists of a series of concrete spandrel beams, with a curtain enclosure of double 6-inch hollow tile.

"The character of the concrete, where it was placed in position with precaution to insure a solid mass, was of such excellent quality that there can be no suspicion of this failure being caused by an inferior quality of cement.

"In one case, I am reliably informed, such carelessness existed in the placing of the concrete as to necessitate the removal of a column after the floors and columns above had been constructed. This reconstruction was accomplished by placing two 2″×4″ shores underneath the girders, one each side, and closely adjoining the column. The concrete was then removed for about half the length of the column and poured anew.

"The form work appears to have been very lightly constructed, the uprights being 2 in. × 4 in., ofttimes spliced midway of their length. The primary cause of the failure seems, from the best information obtainable, to have been the collapse of the form work of the roof while the roof was being cast. The weight of the roof impinging on the fifth floor incurred a condition which the construction was unable to resist. The fifth floor was a little over 2 weeks old at the time of the accident. The constantly increasing load was sufficient to destroy each of the succeeding floors and to carry down with them all the interior construction.

"The building was designed to carry a live load of 40 pounds per square foot. The writer was unable to learn that there was any load even approximating this on the floors at the time of the accident. A calculation of the strength of the girders of the first floor in accordance with the parabolic theory shows that there would exist a fiber stress of about

1,100 pounds per square inch on the concrete. The building laws of San Francisco place a limitation of 450 pounds per square inch on concrete under such a condition as this.

"It will be well to note that an attempt in construction has been made here that is not sanctioned by the most advanced constructors; that is, the elimination of transverse girders and the placing of entire dependence for horizontal transverse forces upon thin slab construction. The entire elimination of haunches or brackets, together with their reinforcement, is a serious sacrifice of lateral stability, particularly so in a location where a structure may be called upon to endure very severe wind conditions.

"A building having steel columns resting one on top of another, with nothing more than a dowel-pin connection, would inspire no feeling of security. It therefore becomes difficult to understand why a concrete structure whose columns contain no more reinforcing material to give strength at the floor level than would be obtained in the above steel connection, should be cited as an example of failure of the reinforced-concrete type. Again, imagine the designer running only a single line of girders between the above steel columns at each floor level in a five-story structure and the faults and weaknesses of the general design are apparent.

"Only the opponent of reinforced-concrete construction would dignify the columns of the Bixby Hotel by classifying them as examples of this type of construction. It is true that they contain some metal, but the amount is so deficient and the placing so careless and inefficient that they can never develop the strength that columns of such dimensions should possess and will have when properly reinforced. The defects existing in the columns would have contributed much less toward the magnitude of the disaster had the proper transverse girders been included in the construction. In fact, it might reasonably be assumed that such girders would have confined the failure to a very small area and might have stopped it at the fifth floor.

"The ruins of the Bixby Hotel show clearly that great care must be taken in the design and execution of such structures. They also confirm the belief that when these precautions have been taken, reinforced concrete contains the merits and security that have been advanced in its favor."

SUMMARY

26. A review of these several failures that have occurred in the last few years illustrates conclusively the great necessity for the exercise of extraordinary precautions in the design and construction of reinforced-concrete buildings. It is well to note particularly that all these failures occurred during the process of erection of the building. In no instance had the building been entirely finished and occupied. It is more than likely, therefore, that even with the existing defects in design and construction that the collapse of the several buildings revealed, they would not have materialized into failure if the concrete had approached its maximum strength. Undoubtedly many of the failures were indirectly due to the fact that the concrete, while it had set up hard, had not attained the full strength that it would in the finished structure, and that it did not possess the factor of safety that reinforced-concrete usually does. While it is bad engineering practice to depend on the assumed factor of safety to take care of defects in design, nevertheless there is no material nor construction that provides such a high factor of safety in the actual conditions over the theoretical assumption as reinforced concrete. The lesson taught by these failures is that too much care cannot be exercised in the designing and supervising of reinforced-concrete structures, and that the materials used must be the best of their respective kinds.

TESTS ON REINFORCED CONCRETE

INTRODUCTION

NEED OF EXPERIMENTAL INFORMATION

1. Concrete has long been used as a structural material under conditions that subjected it to compressive stresses only. Hence, study was at first directed chiefly toward its properties in compression, and little was learned of its other properties. In the last 20 years, however, the type of construction known as reinforced concrete has rapidly come into use. Besides concrete, steel is used in this construction. The steel consists of rods comparatively small in cross-section, and these are so placed that they will provide the necessary resistance most advantageously. The design of reinforced-concrete members involves not only the properties of steel in tension and of concrete in compression, but also the properties of concrete in tension, shear, and adhesion, and the question of how the two materials with properties so dissimilar will act when used together. So rapidly has this construction grown into favor with structural engineers that the experimental knowledge has not been able to keep pace with the demand for it, and as a result designers have made assumptions far beyond the facts of experimental verification.

During the past 5 or 6 years, however, there has been a decided reaction against this practice, and this reaction has given rise to an active and organized demand among

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engineers for definite information backed, not by generalizations from previous knowledge of homogeneous materials, but by actual tests of reinforced concrete.

CLASSIFICATION AND FUNCTIONS OF TESTS

2. Classes of Tests.—There are two general classes of tests for reinforced concrete, namely, *field tests* and *laboratory tests.*

Laboratory tests, as the name implies, include tests made in the laboratories equipped with special machines for testing structural materials.

Field tests include tests made in the field, usually at the site of a building and often on the building itself.

3. Functions of Field Tests.—Field tests usually consist in: (1) the loading to failure of a single isolated structural member, generally a beam or a panel, or slab, similar in all details to a corresponding member to be used in some structure; and (2) the loading of a structural member in a completed structure to an amount equal to or greater than the working load specified in the design.

In either case, the main purpose is that definite assurance may be had that the particular design will do the required duty under the given conditions. The testing of a portion of a completed structure also furnishes a check on the work of the contractor, as well as that of the designer. In order that the check may be entirely satisfactory, however, the test load should be at least twice as great as that specified in the design.

Occasionally, field tests are made to secure data and views for advertising the claims of patented types of construction.

4. Functions of Laboratory Tests.—The main purpose of laboratory tests is to obtain information on which may be based safe and economical design. Occasionally, such tests are made to obtain advertising matter or to check the efficiency of a given design, but even these add appreciably to the definite data available for establishing fundamental principles. Laboratory tests are necessarily limited to isolated members.

5. Comparative Value of Field and Laboratory Tests.—An argument against the value of laboratory tests made by a decreasing group of engineers is that the results obtained from the test of an isolated member are not representative of the possibilities of a similar member when an integral part of a monolithic and continuous construction. These engineers claim further that the strength indicated by a field test, especially of a portion of a completed building, is a better guide to the safe design of other similar structures.

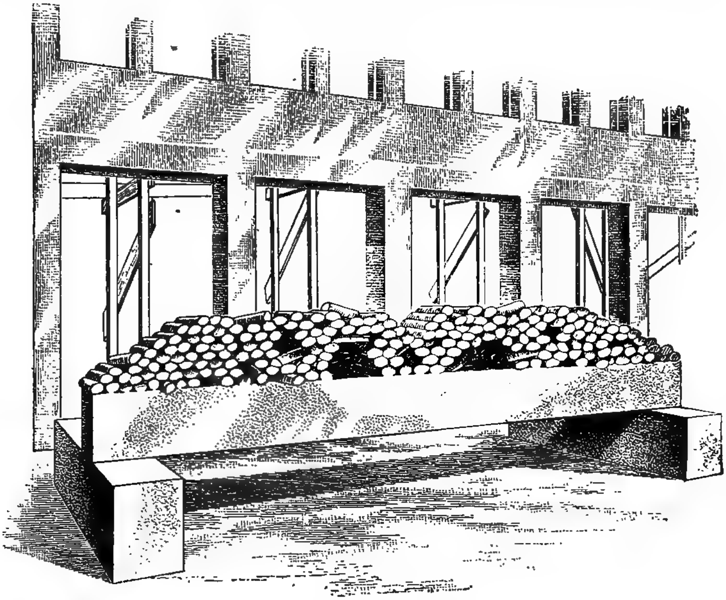


FIG. 1

It is now generally accepted that field tests are apt to be misleading because of the lack of specialized experience on the part of the persons making such tests. Also, the impossibility of determining exactly the conditions of the tests vitiates their conclusions. Of these faults, the last is by far the most important. For example, the usual manner of loading consists in piling pig iron, bags of sand, sheets of metal, etc. upon the beam or slab to be tested, as shown in Fig. 1. Even with careful arrangement of such loads, there is abundant probability of arch action in the loading material. The load in such a case, instead of being uniformly distributed as assumed, is concentrated near the ends, a condition requiring decidedly less resisting moment. Again, it is impossible to ascertain definitely the rigidity of the supports. In one case, complete end restraint may add much to the strength, while in a second test, which to the inexperienced eye differs little as to conditions, it may be a neglible element. Furthermore, the variation in quality of supposedly identical mixtures of concrete, which is great enough in well-regulated laboratories, is subject to much wider variation in the field. For these reasons, it has become accepted that field tests, while of value in demonstrating in a general way the possibilities of reinforced concrete or in checking the quality of the design and construction of a particular member, have little value as data upon which to base deductions for the complete solution of future problems.

6. Classes of Laboratory Tests.—The needs of reinforced-concrete designers not only have caused increased interest in compression tests of plain concrete, but have necessitated the following additional classes of tests: (1) Concrete in pure shear; (2) bond, or adhesion, between concrete and encased steel; (3) unreinforced-concrete beams; (4) reinforced-concrete beams; (5) unreinforced-concrete columns; and (6) reinforced-concrete columns.

In this Section the methods of making these tests will be discussed. The apparatus used will not be fully described, as it is of secondary importance and differs in the various laboratories throughout the country. The object of this Section is to discuss the deformations that take place in concrete structures when loaded and to point out the causes of failure of the various members, so that in practice the engineer may better understand how to guard against such failures.

LABORATORY TESTS

PROPERTIES OF CONCRETE IN PURE SHEAR

7. Methods of Making Tests.—A **shear test** consists in subjecting material to opposite equal forces acting in nearly the same plane. Three early forms of tests are shown in Fig. 2 (*a*), (*b*), and (*c*), in all of which the load was applied over the full width of specimen. Later methods of making these tests are shown in Figs. 3 and 4. In making shear tests, only the maximum load has usually been observed.

8. Data From Shear Tests.—The earlier tests made on specimens, as shown in Fig. 2, indicated an ultimate shearing strength of 10 to 30 per cent. of the crushing strength of the same quality of concrete. Later tests made at the Massachusetts Institute of Technology, in 1905, and the Engineering Experiment Station of the University of Illinois, in 1905, 1906, and 1907, showed conclusively that flexure, or bending, and not pure shear must have caused the specimens of these earlier tests to fail, and that the strength of concrete in pure shear is higher than was at first reported.

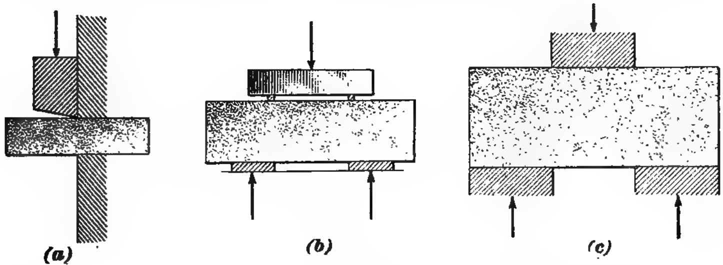


FIG. 2

The tests at the Massachusetts Institute of Technology were made on cylindrical specimens 5 inches in diameter. Two kinds of tests were made at the University of Illinois. The first consisted in shearing a short beam, as shown in Fig. 3. The beam was clamped at the ends to give it additional

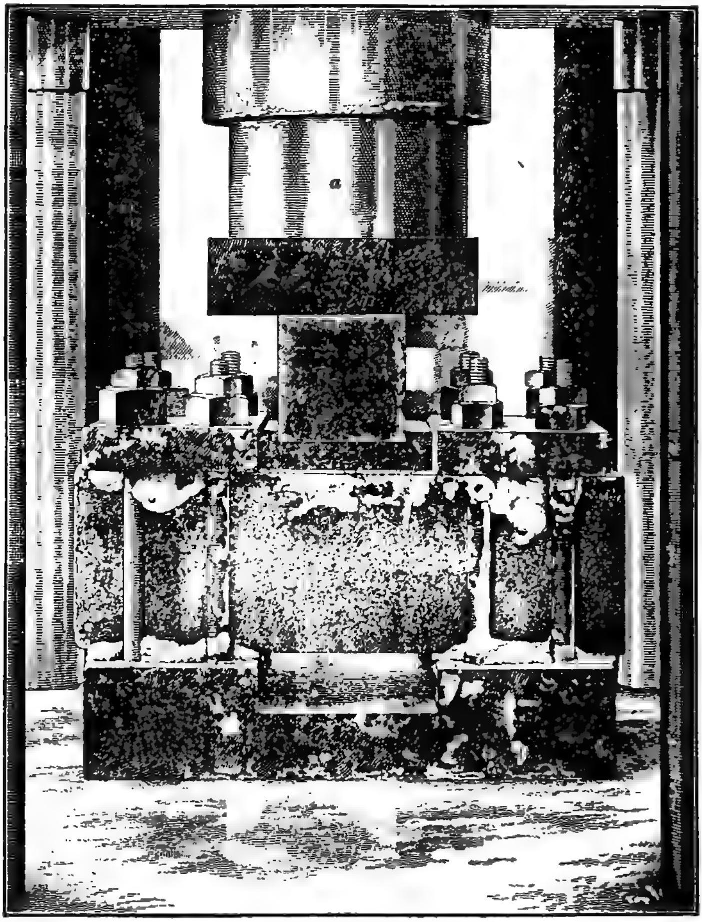


Fig. 3

resistance against bending. The load, of course, was applied with a testing machine, part of which is shown in the illustration at *a*. The beam was bedded in plaster of Paris, as shown, to insure uniform pressure. The second method of determining shear employed at the University of Illinois was to punch a hole in a concrete plate, as shown in Fig. 4. Fig. 5 illustrates the way in which the plates subjected to the punching test failed. The peculiar splitting of the outside into four

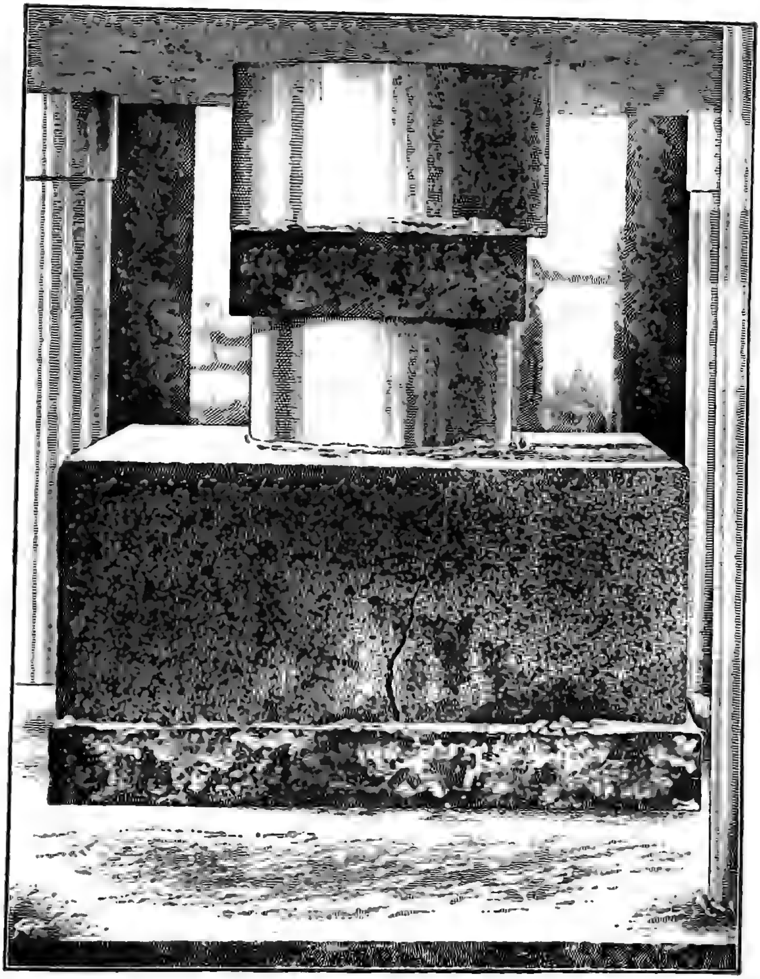


FIG. 4

quadrants is characteristic. It is practically a bursting due to the central core being in compression and bulging. To minimize this effect, the perimeter was strengthened by increasing its thickness, as shown in one test piece at *a*, Fig. 5. In other instances, the perimeter was strengthened by inserting metal reinforcement.

9. The results of the tests made at these two institutions are summarized in Tables I and II. Table I gives the results of the tests at the University of Illinois. Column 1 gives the condition, or form, of the specimen. It will be observed that plain plates in which a hole was punched were first used; then recessed blocks, as shown at *a*, Fig. 5; then recessed blocks that were reinforced around the perimeter to assist in preventing them from bursting; and, finally, specimens in the form of restrained beams, as shown in Fig. 3, were employed.

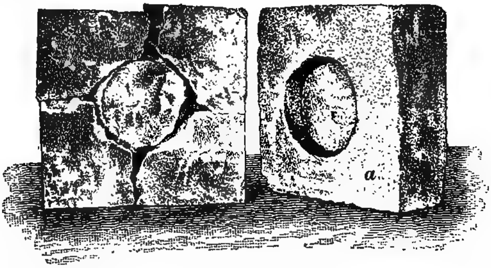


FIG. 5

Column 2 gives the date of the test, and column 3 gives the mixture of concrete used. The method of storing the specimens from the time they were made until they were tested is given in column 4. Column 5 gives the number of specimens of each kind that were tested.

Column 6 gives the strength of the concrete in shear. The compressive strength of the concrete is also given in order that the shear may be compared with this value. The total shear on any specimen just as it was about to fail was taken from the readings of the testing machine. The area sheared in the case of the punch is, of course, the circumference of

TABLE I

SUMMARY OF SHEAR TESTS

(*University of Illinois*)

Form of Specimen	Year	Kind of Concrete	Method of Storing	Number of Tests	Strength Pounds per Square Inch			Ratio of Shear to Compression	
					6			7	
					Shear	Compression		Cube	Cylinder
1	2	3	4	5		Cube	Cylinder		
Plain plate.........	1905	1–3–6	Air	9	679	1,230		.55	
	1905	1–3–6	Water	7	729	1,230		.59	
	1906	1–3–6	Damp sand	4	905	2,428	1,322	.37	.68
	1906	1–3–6	Damp sand	1	968	1,721	1,160	.56	.83
	1906	1–2–4	Damp sand	5	1,193	3,210	2,430	.37	.49
Recessed block.....	1905	1–3–6	Air	17	796	1,230		.65	
	1905	1–3–6	Water	6	692*	1,230		.56	
	1905	1–3–6	Water	5	879	1,230		.71	
	1906	1–3–6	Damp sand	4	1,141	2,428	1,322	.47	.86
	1906	1–3–6	Damp sand	1	910	1,721	1,160	.53	.78
	1906	1–2–4	Damp sand	5	1,257	3,210	2,430	.39	.52
Reinforced recessed block...........	1905	1–3–6	Air	4	1,051	1,230		.85	
	1906	1–3–6	Damp sand	4	1,821	2,428	1,322	.75	1.38
	1906	1–3–6	Damp sand	1	1,555	1,721	1,160	.90	1.34
	1906	1–2–4	Damp sand	5	2,145	3,210	2,430	.67	.88
Restrained beam ...	1906	1–3–6	Damp sand	4	1,313	2,428	1,322	.54	.99
	1906	1–3–6	Damp sand	1	1,020	1,721	1,160	.59	.88
	1906	1–2–4	Damp sand	6	1,418	3,210	2,430	.44	.58

* Specimen injured in removing the forms.

the punch multiplied by the thickness of the concrete. The total shear divided by the area sheared gives the ultimate unit shear, which is the value given in the table. The compression tests were made by compressing cubes and cylinders of concrete to the point of failure and obtaining the unit compressive stresses. The compression values obtained from the cubes differ from those obtained from the cylinders because the cylinders were longer in proportion to their diameter than the cubes were in proportion to their side.

TABLE II

SUMMARY OF SHEAR TESTS

(Massachusetts Institute of Technology)

Kind of Concrete 1	Method of Storing 2	Shearing Strength Pounds per Square Inch 3			Crushing Strength Pounds per Square Inch 4	Ratio of Shear to Compression 5
		Maximum	Minimum	Average		
1-2-4	Air	1,630	960	1,310	2,070	.63
1-2-4	Water	2,090	1,180	1,650	2,620	.63
1-3-5	Air	1,590	890	1,240	1,310	.95
1-3-5	Water	1,380	840	1,120	1,360	.82
1-3-6	Air	1,450	950	1,180	950	1.24
1-3-6	Water	1,200	1,030	1,120	1,270	.88

For this reason, the line of failure of the cylinder was not directly across the cylinder, but was at an angle, and was therefore not due alone to compression but the combined effect of compression and shear. This same characteristic has been noted in crushing a cylinder of other materials, as wood and cast iron.

Column 7 gives the ratio of the values of unit shear to these of unit compression. This ratio is useful, because if a certain kind of concrete has been tested only in compression, an approximate idea of its unit shearing strength may be obtained by multiplying its unit compressive strength by the correct value obtained from column 7 in Table I.

10. Table II gives the results of shearing tests made at the Massachusetts Institute of Technology. Column 1 gives the mixture of concrete; column 2, the method of storing; and column 3, the shearing strength. The different specimens tested, of course, varied somewhat in their strength because of slight differences in making the concrete and other conditions beyond the control of the operator. Therefore, column 3 is divided into three parts, giving maximum, miminum, and average values for these results. Column 4 gives the crushing strength of the concrete, and column 5, the ratio of the shear to compression.

11. The shearing strength of concrete is not now used directly in reinforced-concrete design, but enters into the discussion as a secondary property. For this reason, an average numerical value for shear is not as necessary as the value for compression, and the former is frequently expressed as a percentage of the latter. The two sets of tests summarized in Tables I and II give a shearing strength equal to 60 to 90 per cent. of the crushing strength of small cubes, while values of 10 to 30 per cent. of the crushing strength were formerly accepted. Even in these later tests, the results are reduced somewhat either by induced tension or by excessive bearing or compression. Hence, the values obtained are probably still lower than the true shearing strength.

BOND BETWEEN CONCRETE AND ENCASED STEEL

BOND TEST AND ITS CHARACTERISTICS

12. Method of Testing.—The usual test to determine the bond between steel and concrete consists in measuring the load necessary to pull a steel rod from an encasing body of concrete. Fig. 6 shows clearly the method of testing. To minimize eccentricity of loading, the bearing end of the encasing concrete was frequently cushioned in freshly mixed plaster of Paris under a small load and permitted to harden, but it now seems evident that the inefficiency of this provision was a prominent factor in the disagreements of results reported in earlier investigations for supposedly similar conditions. The use of a spherical bearing block, as shown in Fig. 6, is a more positive method of eliminating eccentricity of pull than is cushioning with plaster of Paris and gives more uniform results. The principal observation has been that of maximum load, although in the more recent series much study has been directed toward the amount of small displacements of the rod relative to the concrete, and to the loads corresponding to these displacements. These small displacements may be measured by means of a measuring apparatus known as a *spherometer*, not shown in the illustration. The load applied is divided by the area of the surface of the rod in contact with the concrete. This quotient is known as the *bond per unit area*, or simply the *unit bond*, and is the basis on which bond tests are compared.

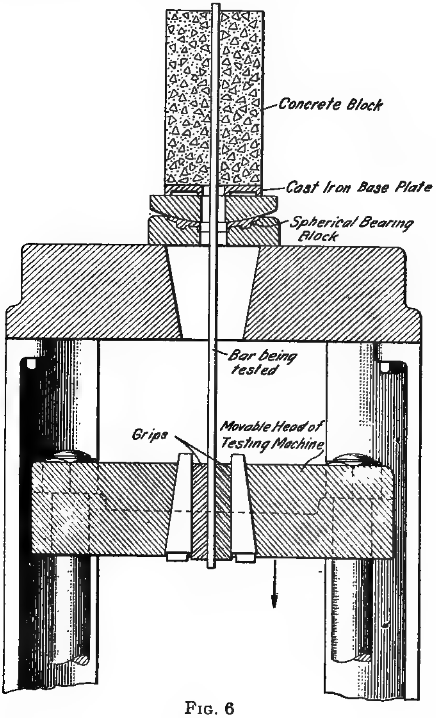


FIG. 6

It should be noted that in a bond test made as here described the concrete next to the bar is in compression; in a reinforced-concrete beam, the concrete adjacent to the rods is usually in tension. Tests made at the University of Wisconsin in 1908 would seem to indicate that the ultimate resistance to slipping is less in the latter case than in the former. The number of tests, however, was too few to justify the acceptance of this conclusion as definite.

13. General Characteristics of a Bond Test. In a bond test of a plain rod, that is a rod without deformed surfaces, there are no visible indications of change as the load is gradually increased to the maximum, although recent exact measurements have shown that there is a small relative motion before the maximum load is reached. When the rod once starts to slip, it slips rapidly, and this can be seen without the use of delicate measuring instruments. It has been found that even after the direct bond of the two materials has been broken, a considerable force is required to continue the motion of the rod, and also that this force seems to remain somewhat constant during a further withdrawal. The term *running friction* is applied to this resistance to relative motion after the direct bond has been broken.

FACTORS AFFECTING THE ADHESION OF CONCRETE AND STEEL

14. The principal factors affecting the bond between concrete and steel are: (1) the yield point of the steel, (2) the shape and size of the bar, (3) the character of the surface, and (4) the quality and age of the concrete. As in other tests involving the properties of concrete, it must be expected that different experimenters will get somewhat divergent results because the quality of concrete is dependent on so many factors.

15. Yield Point of Steel.—The total force required to break the bond of undeformed bars cannot much exceed the yield point of the encased rod. This is not definitely shown by any one series of tests, but is a conclusion derived from a general consideration of various recent tests This fact is explained on the ground that after the yield point is passed, the reduction in cross-section of the steel becomes more pronounced and the bond is broken by the contraction of the steel *away* from the concrete rather than by longitudinal motion *past* the adjoining concrete. In other words, the conditions of adhesion beyond the elastic limit and adhesion before the elastic limit are decidedly different. This distinction serves to divide bond tests of plain rods into two general classes, each of which should be studied by itself and comparisons between which should not be made the basis for too general deductions.

TABLE III

SUMMARY OF ADHESION TESTS ON SMOOTH BARS

Name of Laboratory	Reference Number	Size of Rod Inches	Kind of Concrete	Encased Length Inches	Unit Bond Pounds per Square Inch
University of Illinois ...	1	½ diameter	1–2–4	6	412
	2	½ diameter	1–2–4	12	404
	3	⅝ diameter	1–2–4	6	465
	4	⅝ diameter	1–2–4	12	414
University of Minnesota	5	¾ diameter	1–3–6	8	420
	6	¾ diameter	1–3–6	17	400
	7	⅜ diameter	1–3–6	8	370
	8	⅜ diameter	1–3–6	10	420
	9	1¼ × ⅜	1–3–6	8	230
	10	1¼ × ⅜	1–3–6	16	270
	11	2 × ¼	1–3–6	8	270
	12	2 × ¼	1–3–6	17	240

16. Effect of Size and Shape of Bar.—Table III gives a summary of the adhesion tests made on smooth bars at the University of Illinois and at the University of Minnesota. All rods used in the tests were plain commercial bars, and the maximum load was less than the elastic limit in all cases. There is no definite information as to the relative quality of the concrete used in the two series of tests. From a comparison of the values in each odd-numbered line with the corresponding values in each succeeding even-numbered line, it may be seen that the unit bond is practically independent of the length of embedment. Furthermore, for the round rods, the unit bond strength is practically constant for the range of diameters from ⅜ to ¾ inch. This may not be true, however, for extremely large rods or for thin wires. The results of tests by both the Illinois and the Minnesota University indicate that for round commercial bars ¾ inch in diameter, an average value of bond for 1–2–4 limestone concrete, under laboratory conditions, is 400 pounds per square inch at 60 days, and that 300 pounds per square inch may be accepted as the ultimate strength on which safe design may be based.

17. Effect of Roughness of Surface.—The condition of the surface of the rod has a decided effect on the bond resistance. At the University of Illinois, cold-rolled shafting having a very smooth surface developed 160 pounds per square inch as compared with 375 pounds per square inch for an ordinary plain commercial bar. Professor Raymond, of the State University of Iowa, found from his tests, in 1904, that coating the rods with oil, graphite, soap, lead, etc. reduced the bond from 50 to 75 per cent. below that of uncoated steel. No tests concerning the effect of slight rusting of the surface of bars have been reported.

18. Effect of Quality and Age of Concrete.—The adhesion of concrete to steel increases with the richness of the concrete, the wetness of the mixture, etc. The available data does not justify definite statements of the rate of increase, but two cases will illustrate actual results. At the University of Illinois, the bond for 1–2–4 concrete was found to be from 10 to 15 per cent. greater than that for a 1–3–6 mixture. At the University of Minnesota, the average bond for ⅜-inch round rods was 310 pounds per square inch for 1–4–8 concrete, 370 pounds per square inch for 1–3–6 concrete, and 410 pounds per square inch for a 1–2–4 mixture.

19. No extensive series of tests has been made to determine the effect on the bond between concrete and steel with reference to the age of the concrete. A few tests were made, however, at the laboratory of the University of Wisconsin, in 1908.

The results of these tests are shown graphically in Fig. 7. The strength of the concrete is given in terms of its strength at 30 days, which is taken as unity. Thus, the figures on the lower line represent the age of the concrete, in days. As an example, assume that it is desired to find the relative strength of the concrete in 90 days. This may be done by looking along the bottom line of the diagram until the figure 90 is reached and then glancing vertically up to the curve and then horizontally back to the left-hand border line. It will be seen that the point arrived at on the left-hand border line is directly between the figures 1.10 and 1.20, say at 1.15. Therefore, the unit bond between concrete and steel at 90 days is 1.15 times the unit bond at 30 days; that is, in 90 days the bond has increased 15 per cent. above that at 30 days.

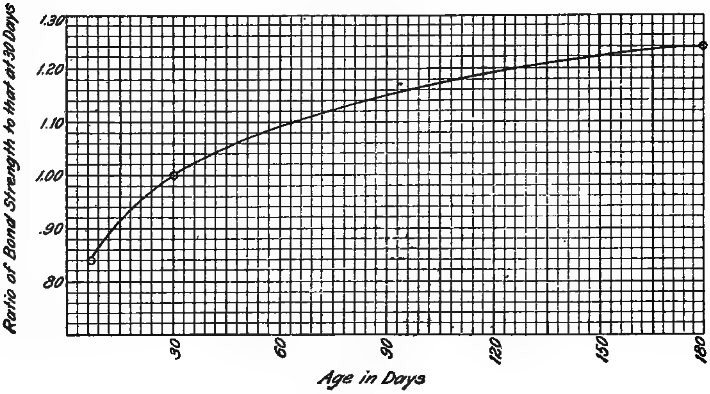


FIG. 7

RUNNING FRICTION

20. It has been noted in a preceding article that the load required to continue the motion of the encased plain rod after the adhesion has been positively broken is a considerable proportion of the maximum load, and that it remains practically constant during a considerable withdrawal. This has been termed running friction. Prof. W. K. Hatt found it to range from 50 to 70 per cent. of the maximum load, and Prof. A. N. Talbot noted a range of 54 to 72 per cent. for commercial bars and one of 32 to 50 per cent. for small cold-rolled shafting with a smooth surface.

These figures have been used as the basis of an argument to prove that the bond of plain steel rods to concrete is due more to the tight gripping action caused by the concrete slightly expanding as it sets than to the direct adhesion of the concrete surface to the steel surface. The few tests so far made, however, do not offer sufficient evidence to establish this view.

BOND TESTS OF DEFORMED BARS

21. The sufficiency of the bond developed by plain bars for all conditions has been a much mooted question. There are on the market many types of deformed bars that are advocated by persons who maintain the insufficiency of the bond of the plain rod. These bars are provided with many projections that furnish positive shoulders against which the concrete can press, and, as would be expected, their resistance to withdrawal is decidedly higher than that of plain rods. As compared with the conditions

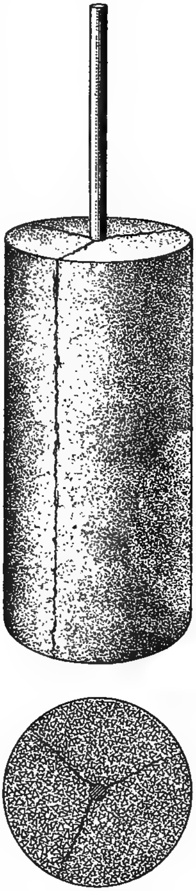


Fig. 8

actually existing in a beam, the usual bond test is probably less favorable to the deformed bar than it is to the plain bar.

In the ordinary bond test of a deformed bar, the maximum load usually corresponds either to the splitting of the encasing concrete, as illustrated in Fig. 8, or to the rupture of the bar in tension outside the encased length. In neither case does the maximum load correspond to the actual rupture of the concrete about the perimeter of the rod. Prof. C. E. Depuy, of Lewis Institute, Chicago, found bond values for deformed bars of 500 to 800 pounds per square inch of embedded surface for 1–2–4 concrete, and Prof. A. N. Talbot reported 300 to 600 pounds per square inch for a mixture of the same proportions. In both series, ultimate failure was due to the splitting of the concrete block. A block of larger cross-section with its increased resistance to splitting would probably have given higher values for the bond test. Since failure did not actually occur by slip, this method of expressing the results in pounds per square inch of embedded surface has no logical foundation, but it furnishes a basis of comparison with similar tests of plain bars and serves to bring out the fact that deformed bars offer, say, double the ultimate resistance to motion that plain rods do.

22. Vibrating Loads.—There is no data available in regard to the effect of repeated, or vibratory, loads on the bond between concrete and steel. It is sometimes claimed that repeated and long continued variation of load decreases the bond, but there is no positive evidence either to support or to discredit this claim.

PLAIN-CONCRETE BEAMS

23. Methods of Testing.—A few tests have been made on concrete beams that are plain, or unreinforced. The methods of testing are identical with those of reinforced-concrete beams, which will be described in detail later. The observations made are maximum load and the deformations of the upper and lower fibers, and the deflection at the center of the span for various loads.

24. Results.—Unreinforced-concrete beams fail suddenly by breaking across normal cross-sections. Such beams are seldom used because of their comparatively low strength and because complete failure occurs without the least previous warning. Prof. A. N. Talbot found a modulus of rupture of 300 to 400 pounds per square inch for 1–2–4 concrete at 60 days. The results of some of the tests made by the United States government are given in *Concrete Beam and Column Design.* The deformation measurements for the upper and lower fibers have brought out clearly that the elongation on the tension side is approximately equal to the corresponding shortening on the compression side. In other words, the elastic properties of concrete in tension up to rupture are much the same as in compression up to an equal unit stress. Before Professor Talbot's test of plain-concrete beams in 1904, the modulus of compression had frequently been published as twice the modulus in tension, a discrepancy due probably to the difficulties of making a pure tension test of concrete and of getting the same quality of concrete in both tension and compression test pieces.

REINFORCED-CONCRETE BEAMS

TESTING AND LOADING

25. The method of testing a reinforced-concrete beam varies slightly with the special purpose of the investigation. The description immediately following applies only to the general features of such tests.

26. The loads are applied by means of a testing machine that, in most American makes, consists essentially of a power-driven apparatus for applying pressure and a weighing apparatus. As shown in Fig. 9, the weighing apparatus is identical in principle with the common platform scales. Of course, the machines used today are much more complicated that the one shown; however, the illustration, which shows the machine arranged for a compression test, will serve to bring out the principle on which testing machines work. At *a* is shown the platform upon which the specimen to be tested is placed. The pulling head *b* is pressed down upon the test piece by means of screws *c*, which are usually power driven. Fig. 10 shows an actual machine as arranged for a beam test.

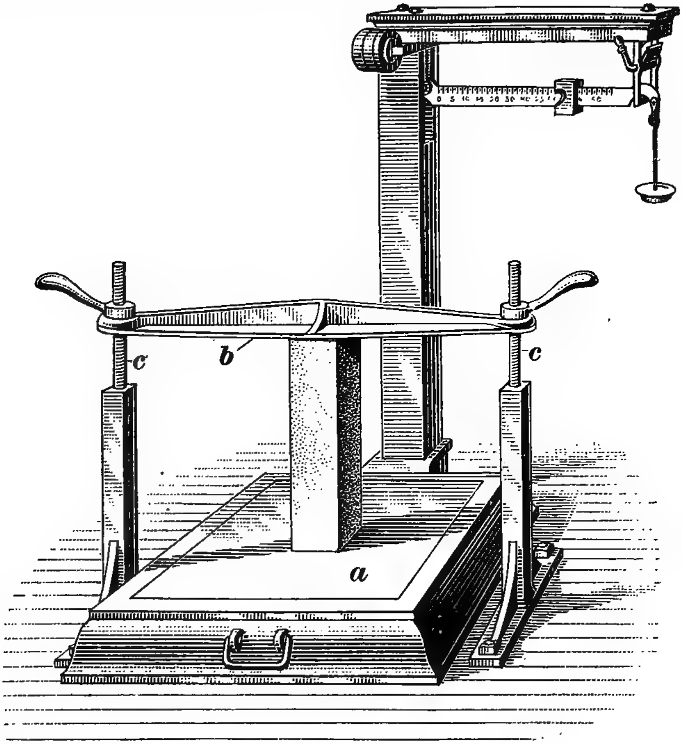


FIG. 9

The beam rests on supports that permit ready longitudinal adjustment of the span as the bottom fibers elongate under the load. In most tests, the load is applied equally at two points symmetrically placed with respect to the middle of the span, usually the $\frac{1}{3}$ points of the span. The purpose of having the load applied there rather than at the center is to

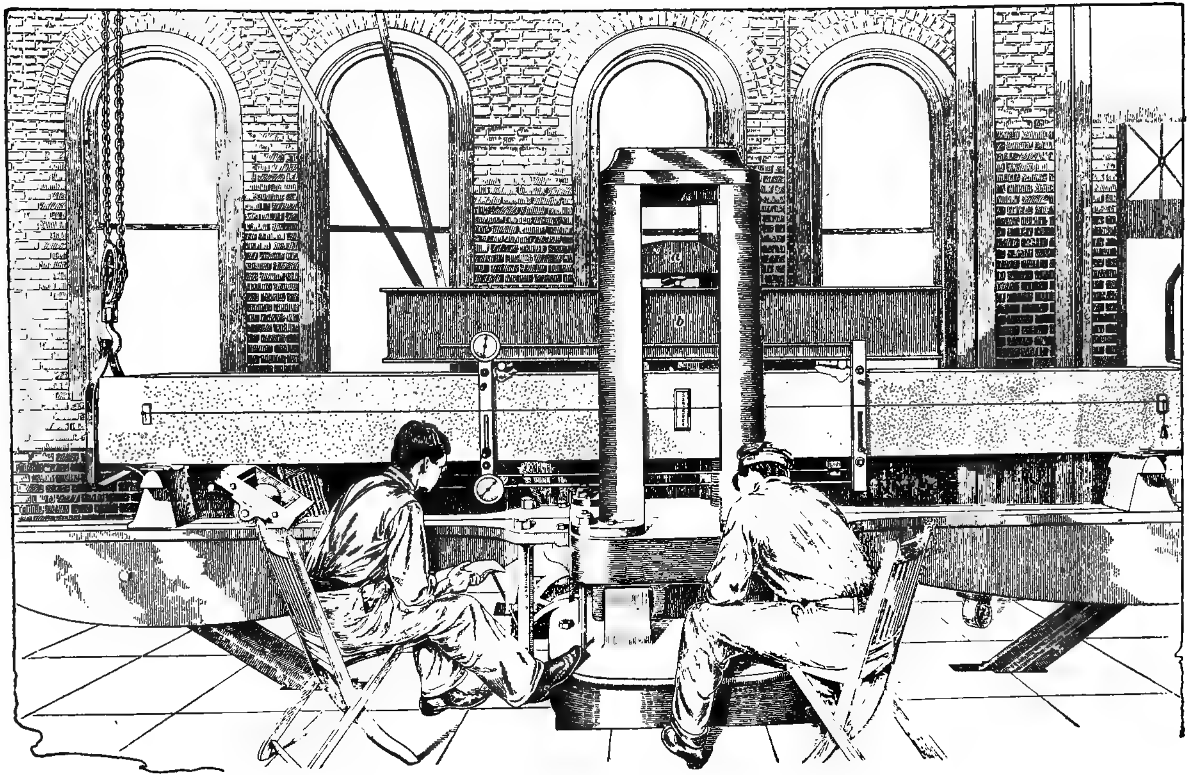
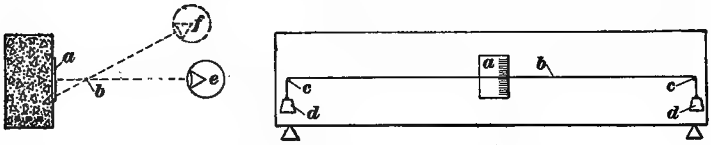


FIG. 10

provide a portion of the span over which the bending moment, and hence the unit fiber deformations, remains practically constant. The load is transferred from the pulling head *a* to the application points by means of a steel **I** beam *b*. Steel rollers at the application points of the load minimize the resistance to free longitudinal deformation of the fibers, and narrow steel plates embedded in plaster of Paris by a small load serve to reduce the local compression action.

DEFLECTIONS AND DEFORMATIONS

27. Deflection Apparatus.—Deflections at the center of the span are usually observed as a check upon the general characteristics of the *load-deformation curves* to be explained later. A common and convenient method of making this observation is to fasten a graduated mirror to one side of the beam at mid-depth and at the center of the span. In front of this mirror is stretched a fine thread that passes over fixed points at the mid-depth of the side of the beam immediately over the supports. This thread is kept at a constant tension by a small weights fastened to the ends. The reading is made by lining up the thread and its reflection and noting the corresponding graduation on the scale. There are more exact methods of obtaining this measurement, but the accuracy of the one described here is entirely sufficient for the purpose of a check on the deformation curves.



(a) (b)

FIG. 11

28. The method of measuring the deflection will be better understood by referring to Fig. 11, in which view (*a*) shows a section of the beam and view (*b*) an elevation. The graduated mirror is shown at *a*, and the string at *b*. The string is held up by two pins *c* located over the points of support. A reading is taken on the graduated scale on the mirror before the load is applied to the beam. As the load increases, other readings are taken, and as the beam sags, the scale moves behind the string, which is stretched tight by the weights *d* at its supports.

The reason for having the scale made on a mirror instead of on steel or wood will be apparent by referring to the sectional view (*a*). This view shows two positions that the observer's eye might assume in looking at the scale. It is evident that in these two positions the string marks different parts of the scale. Therefore, different experimenters might read the scale differently, according to the position of the eye. This discrepancy is known as *parallax*, and must be guarded against in making all measurements in a concrete laboratory. To avoid the error of parallax, every observer, instead of looking at an angle, must look directly at the mirror. When the observer looks perpendicularly at the mirror, as shown by the eye at *e*, and not slantwise, as shown at *f*, the string itself will just cover its own image in the glass and hide it from the eye of the observer. Therefore, the observer places his eye so that the string directly covers its own image in the mirror and then reads the scale directly under the string.

29. Measurement of Horizontal Deformations. Fig. 12 shows one type of device for measuring the compression at the top and the expansion at the bottom of a beam when loaded. This phenomenon is the **horizontal deformation.** The measuring device consists of two yokes *a* and *b*, each fastened to the beam by means of two pair of thumbscrews *c*, one set of which is placed opposite the reinforcing rods and the other an inch or so below the top of the beam. These yokes are usually placed just without the points where the load is applied as shown in Fig. 10. The horizontal distance between the contacts of the two yokes is known as the *gauge length*. On one yoke are mounted four brass rollers *e*, two above and two below. Connected with each roller is a pointer *f* rotating in front of a graduated circle *d*.

On the second yoke, in positions corresponding to the rollers on the first yoke, are attachment pins *h*. The gauge bars *g* extend between the yokes, being fastened to the pins at one end and resting on the brass rollers at the other. As the upper fibers compress and the lower fibers elongate, under the action of a load, the relative motion of the two yokes is indicated by the motion of the pointers on the graduated dials. With an attachment known as a *vernier*, each dial can be read to .0001 inch.

30. Additional Observations.—The measured readings usually taken in beam tests are deflections and deformations corresponding to various loads, but fully as important in the correct interpretation of the results are the general observations made as to the location and appearance of the first minute crack and to the nature of the development of the more important subsequent cracks. Of special importance is a complete and accurate description of the beam just at the development of the maximum load or immediately after it. These conditions furnish the strongest evidence as to the causes underlying the limitation of the maximum load and failure of the beam, and these causes form the basis of the classification of the failures into groups for a proper discussion of their features.

INTERPRETATION OF RESULTS

31. Computation of Unit Deformation in Steel and in Concrete.—The measurements taken on the two yokes, or *extensometer device*, shown in Fig. 12, give the relative motion of the two cross-sections at two points. The unit deformations in the steel reinforcement and in the extreme fiber of the concrete in compression are computed graphically. The method is based on the assumption that a normal cross-section that is plane before bending remains plane after bending. This assumption has been checked by a small number of tests made by Prof. A. N. Talbot, and is in line with the accepted theory for homogeneous beams.

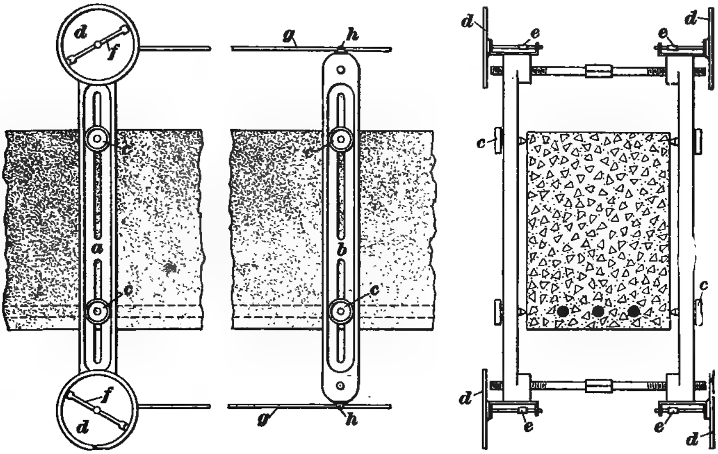
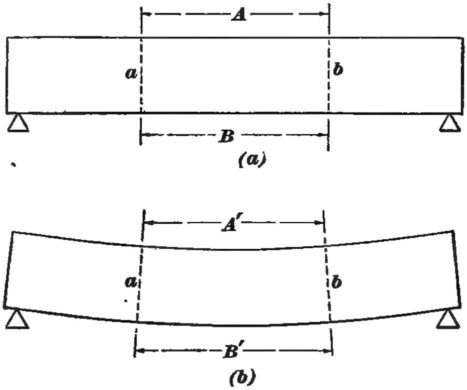


FIG. 12



(a)

(b)

FIG. 13

The measurement taken by the two yokes will be understood by referring to Fig. 13. The beam before it is loaded is shown in (*a*), the position of the two yokes being shown at *a* and *b*. The reading on the top extensometer will be A, and that on the bottom one will be B. After the beam is loaded, it sags, as shown in an exaggerated manner in (*b*). The top extensometer will then measure the distance A', and the bottom extensometer will measure the distance B'. Since the beam sags, the top of the beam, which is in compression, shortens and the bottom of the beam, which is in tension, lengthens. The total shortening between the two yokes at the top is equal to $A - A'$, while the total lengthening between the two yokes at the bottom is equal to $B' - B$. The unit shortening, that is, the shortening per unit of length, is equal to $\frac{A-A'}{A}$, while the unit extension, or lengthening, is equal to $\frac{B'-B}{B}$.

A slight correction must be made in the calculations owing to the fact that A' and B' are not measured directly on the beam, but a little above and below it, and therefore their readings are somewhat exaggerated. This correction is made as shown later.

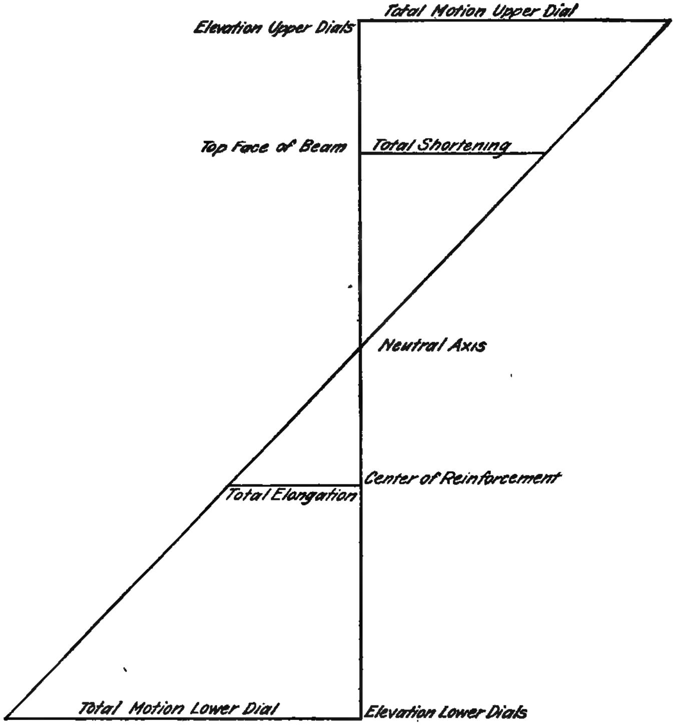


FIG. 14

32. The method used in practice to find the location of the neutral axis is as follows: Referring to Fig. 14, the vertical line represents a normal cross-section of the beam

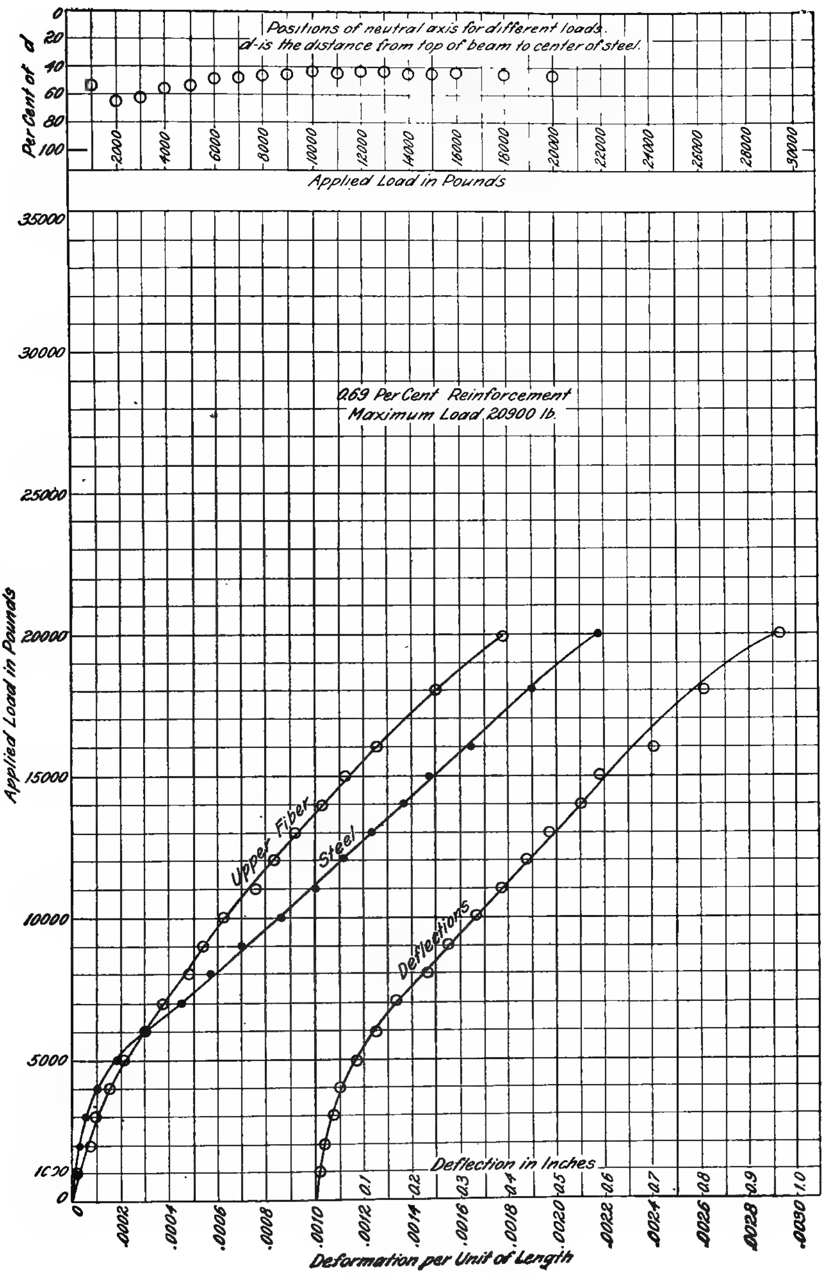

Positions of neutral axis for different loads.
d-is the distance from top of beam to center of steel.
Per Cent of d
0
20
40
60
80
100
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
26000
28000
30000
Applied Load in Pounds
0.69 Per Cent Reinforcement
Maximum Load 20900 lb.
Applied Load in Pounds
35000
30000
25000
20000
15000
10000
5000
1000
0
Upper Fiber
Steel
Deflections
Deflection in Inches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
.0028
.0030
Deformation per Unit of Length

FIG. 15

under no load, say at the point where the yoke *a*, Fig. 13 (*a*), is located. On this line are located, according to some scale, points corresponding in elevation to the upper and lower dials on the yoke, as well as the center of the steel reinforcement and the top face of the beam. At the elevation of the upper dials there is laid off horizontally in one direction the motion indicated on these dials; and similarly at the elevation of the lower dials there is laid off in the opposite direction, horizontally, the motion indicated on the lower dials. A straight line joining the outside ends of these two horizontal lines represents graphically the normal cross-section after bending. Then the horizontal intercepts between this inclined line and the vertical line at the elevation of the steel and at the elevation of the top of the beam represent the total deformations in the corresponding fibers in a length equal to the gauge length. The unit deformations are computed by dividing the total deformation by the gauge length. The point at which the inclined line intersects the vertical line gives the position of the neutral axis, for since there is no deformation at this point, there is no stress. The position of the neutral axis is usually measured from the compression, or upper, face of the beam, and its distance from this face is expressed in percentage of the depth of the beam from the top surface to the center of gravity of the steel reinforcement.

33. Graphical Presentation of Data.—Fig. 15 shows a characteristic **load-deformation curve** for a beam not having a large amount of reinforcement. The horizontal scale gives the average unit elongation in the steel and the unit shortening in the upper fibers of the concrete in the gauge length used. The vertical scale is the total applied load, in pounds. In comparing beams of different breadths and depths the vertical scale, instead of being made equal to the total load in pounds, is sometimes made equal to the bending moment of the applied load divided by the product of the breadth and the square of the depth. The load-deflection curve is usually placed on the same diagram as the load-deformation curve to permit of better comparison. The vertical scale is the same, but a different horizontal scale is used, giving the deflections at the center of the span in inches. The upper part of the diagram shows graphically the position of the neutral axis at different loads.

34. As an example of the use of this diagram, let the condition of the beam that the diagram represents be investigated when the total load is, say, 12,000 pounds. This value of 12,000 pounds is read off on the vertical scale on the side. By following a horizontal line from the 12,000-pound mark on the vertical scale to where this line cuts the curve marked *Upper fiber*, and descending in a vertical line from the point of intersection to the horizontal scale, the point a little beyond .0008 is reached. This value is the unit deformation in the concrete in the top of the beam; that is, 1 inch of concrete in the top of the beam before loading would be compressed to $1-.0008=.9992$ inch after a load of 12,000 pounds has been put on the beam. Likewise, the unit elongation of the steel under the same load is about .0011. The total sag, or deflection, of the beam under the load of 12,000 pounds is also found by the same process, using, of course, the deflection curve. The deflection for this load is seen to be almost .45 inch.

The method of locating the position of the neutral axis has been described. It has been found that the position of this axis varies with the load on the beam. This variation is shown in the upper part of Fig. 15. Here, the horizontal scale gives the load on the beam, the same as the vertical scale in the lower part of the diagram. The vertical scale gives the *Per Cent. of d;* that is, the distance from the top of the beam to the neutral axis expressed as a percentage of the distance from the top of the beam to the center of the steel reinforcement, which is its effective depth. Thus, in the diagram, for a load of 12,000 pounds, it will be seen that the distance from the top of the beam to the neutral axis is only a little over 40 per cent. of the effective depth; whereas, with a load of 2,000 pounds, the distance from the top of the beam to the neutral axis is over 60 per cent. of the effective depth.

35. Four Stages of Flexure.—The general characteristics of flexure of reinforced-concrete beams were first correctly reported by Prof. A. N. Talbot, in 1904. In Bulletin No. 1 of the Engineering Experiment Station of the University of Illinois, he says:

"A study of the tests and of the load-deformation diagram shows four stages of flexure during the progress of the loading. The general phenomena attending these four stages are as follows: Through the first stages, as the load is applied, the action of the beam and the changes in the deformation of the upper and lower fibers are similar to those in a plain-concrete beam, modified, of course, by the metal reinforcement; and the resistance of the tensile stresses of the concrete at the bottom of the beam is plainly apparent. When a load of 3,000 to 4,000 pounds is reached (probably equivalent to about 250 pounds per square inch tension in the extreme fiber of the concrete for its share of the applied load and 350 pounds per square inch when the weight of the beam is considered), the second stage begins. The steel elongates more rapidly with the application of the load, there is a similar increase in the compression of the concrete, the neutral axis rises, and there is a marked change in the character of the load-deformation diagram. While no cracks in the concrete at the under side of the beam are visible to the naked eye at this state, apparently much of the tensional value of the concrete has been lost. This stage may be called the *readjustment stage.* During the third stage, the increments of the deformation of the steel are closely proportional to the increments of the load, as shown by the straight line of the load-deformation diagram, and the compressive deformations generally approximate a straight line. During this stage, fine vertical cracks appear, generally quite numerous and well distributed along the middle third of the length of the under side of the beam, and gradually grow more distinct; but this appearance is not accompanied by any change in the character of the load-deformation diagram. This stage continues until a point at or near the maximum load is reached, when the fourth stage, or stage of failure, begins." For the first three

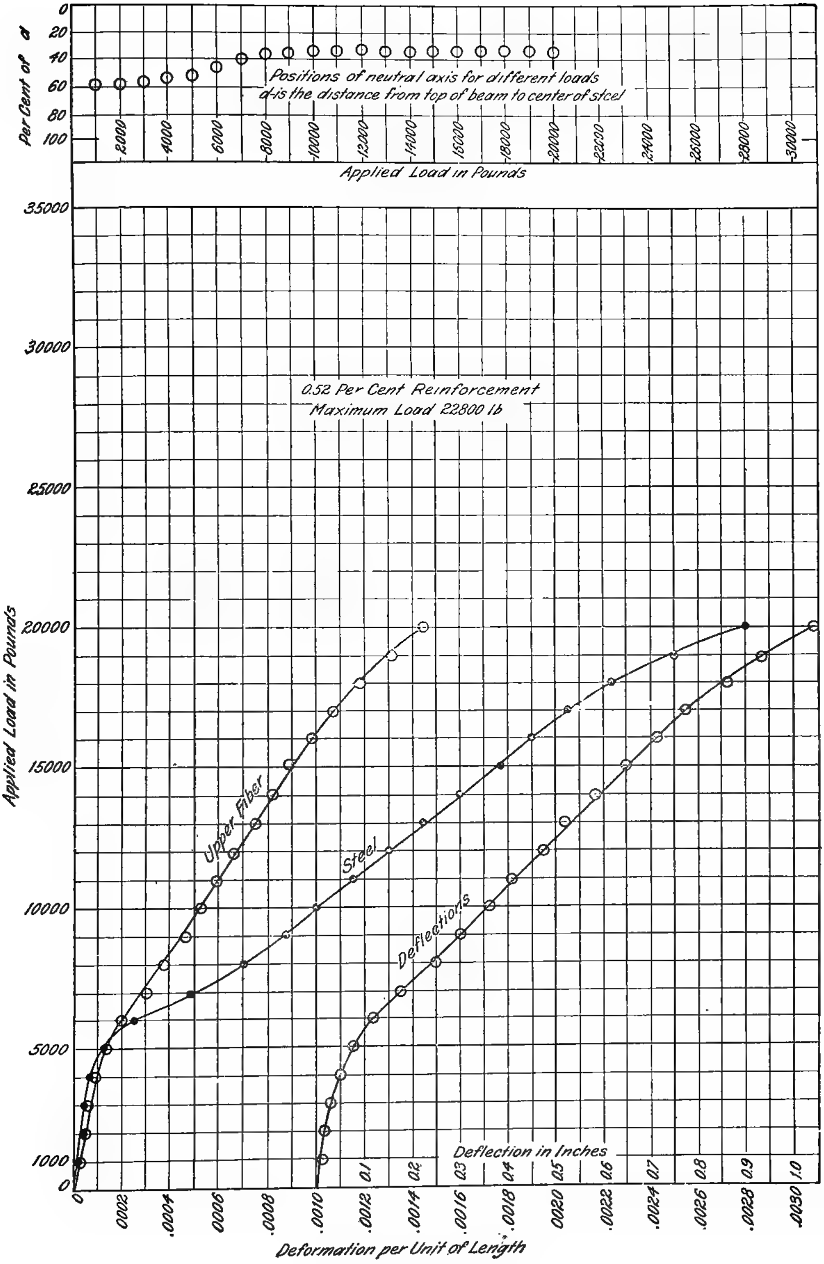

Per Cent of d
0
20
40
60
80
100
Positions of neutral axis for different loads
d-is the distance from top of beam to center of steel
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
26000
28000
30000
Applied Load in Pounds
35000
30000
25000
20000
15000
10000
5000
1000
0
Applied Load in Pounds
0.52 Per Cent Reinforcement
Maximum Load 22800 lb
Upper Fiber
Steel
Deflections
Deflection in Inches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
.0028
.0030
Deformation per Unit of Length

FIG. 16

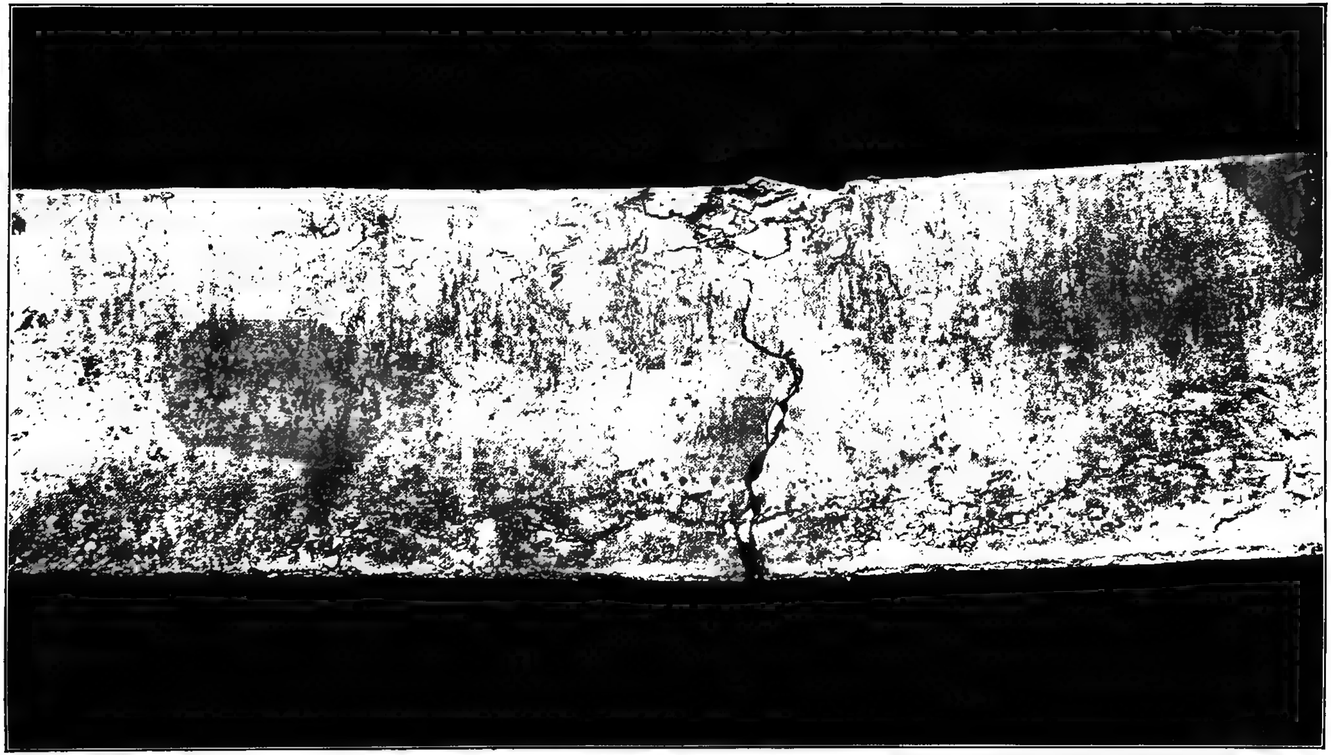


FIG. 17

stages, the general features of flexure are practically the same for all beams, irrespective of the amount and disposition of the reinforcement or the quality of the concrete. In the fourth and final stage, however, there are several distinct classes.

36. Classes of Failure.—Failure of a reinforced-concrete beam occurs when the weakest element has reached the limit of its resistance. Depending on the amount and disposition of the reinforcement, the quality of the concrete, the relation of the span length to the depth, and the distribution of the load, a beam may fail: (1) by the steel yielding at or near the section of maximum bending moment; (2) by the concrete crushing at the same place; and (3) by a diagonal tension failure of the concrete at a place where the shear is large.

These types of failure are frequently referred to respectively as *tension failures*, *compression*, or *crushing*, *failures*, and *diagonal tension failures*. Formerly, the third class was commonly called a *shearing failure*, but it is now more frequently designated by the more exact term diagonal tension failure.

37. Tension Failure.—Fig. 16 shows a characteristic load-deformation diagram, similar to the one shown in Fig. 15, for a beam failing by tension in the reinforcement. Fig. 17 shows a view of such a beam after the test. The visible characteristic, as seen from Fig. 17, is the more rapid development of one or several of the vertical cracks that appeared during the third stage of flexure. In the diagram, the long steel-deformation curve approaching the horizontal is more or less representative of this class of failure. The explanation that may be made for the characteristic path of this curve is that the steel reinforcement has reached its yield point and is stretching with but little increase in the stress, and hence, with but little increase in the load on the beam. The raised position of the neutral axis shown in the top part of the diagram is characteristic of beams failing by tension in the steel. This change of the neutral axis means either a decrease in the area of the concrete resisting compression or an increase in the unit stress in the extreme fiber, as the total compressive

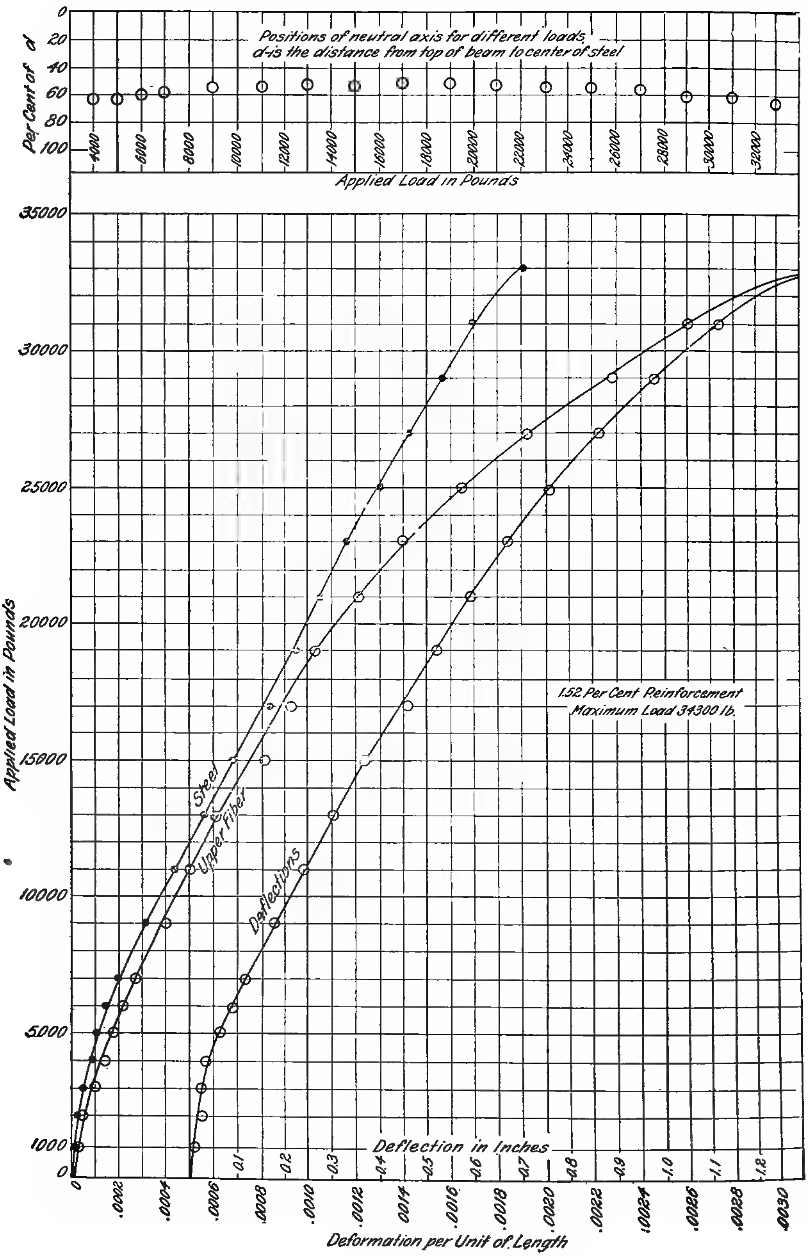

Positions of neutral axis for different loads.
d-is the distance from top of beam to center of steel
Per Cent of d
0
20
40
60
80
100
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
26000
28000
30000
32000
Applied Load in Pounds
Applied Load in Pounds
35000
30000
25000
20000
15000
10000
5000
1000
0
Steel
Upper Fiber
Deflections
1.52 Per Cent Reinforcement
Maximum Load 34300 lb.
Deflection in Inches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
.0028
.0030
Deformation per Unit of Length

FIG. 18

stress is almost constant. This increased stress in the concrete soon causes crushing. It is to be noted, however, in this connection that the real cause of failure is not the crushing of the concrete, but rather the increased deformation in the steel after the yield point. The concrete crushed after the steel had stretched.

38. Compression Failure.—In Fig. 18 is shown a characteristic diagram for a beam that failed by crushing of the concrete. Fig. 19 shows such a beam after the maximum load had been applied. In beams having a larger amount of reinforcement than those failing by tension, failure may be due to crushing out of the concrete at the top when the maximum load is about reached. As distinguished from a tension failure, there is in this case no marked development of the vertical cracks at the bottom, and the first visible evidence of failure is the buckling of the top fibers at or near the section of maximum bending moment. On the stress-deformation diagram this form of failure is usually clearly indicated on the upper fiber curve by the increased falling away from the approximate straight line, as can be seen in Fig. 18. It is also indicated by the lowered position of the neutral axis as the maximum load is approached. This change of position is not so marked as in a tension failure and comes earlier relative to the maximum load.

39. Diagonal Tension Failure.—In beams so loaded as to develop considerable vertical shear, that is, deep beams on short spans, and in which proper attention has not been paid to diagonal or vertical reinforcement, failure will probably be due to the development of one or more diagonal cracks that extend upwards and inwards toward the center of the span, as shown in Fig. 20. The development of these cracks may be gradual, with visible evidence of approaching failure, or it may be sudden, with little or no warning. In addition to the diagonal cracks, this type of failure may be accompanied by a splitting along the plane of the bars from the main diagonal crack outwards, especially if the failure is sudden and not gradual. Such splitting, however, is depend-

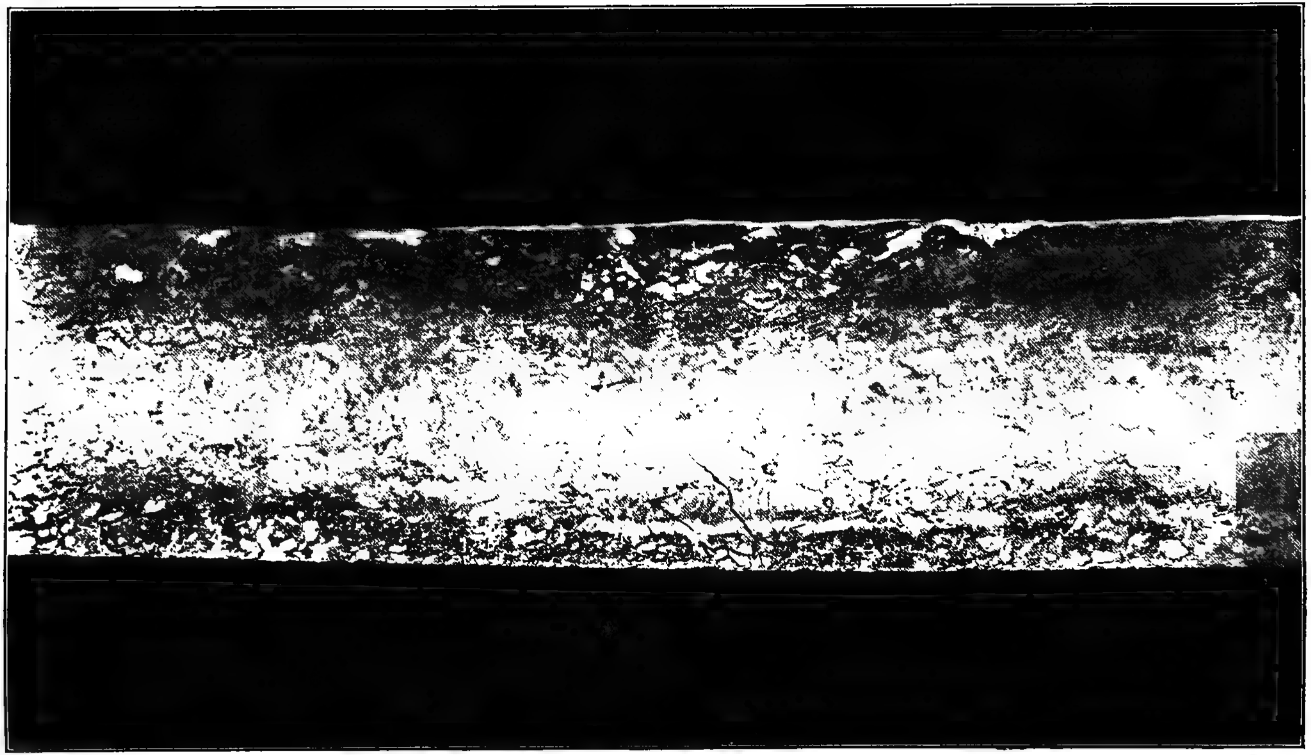


FIG. 19

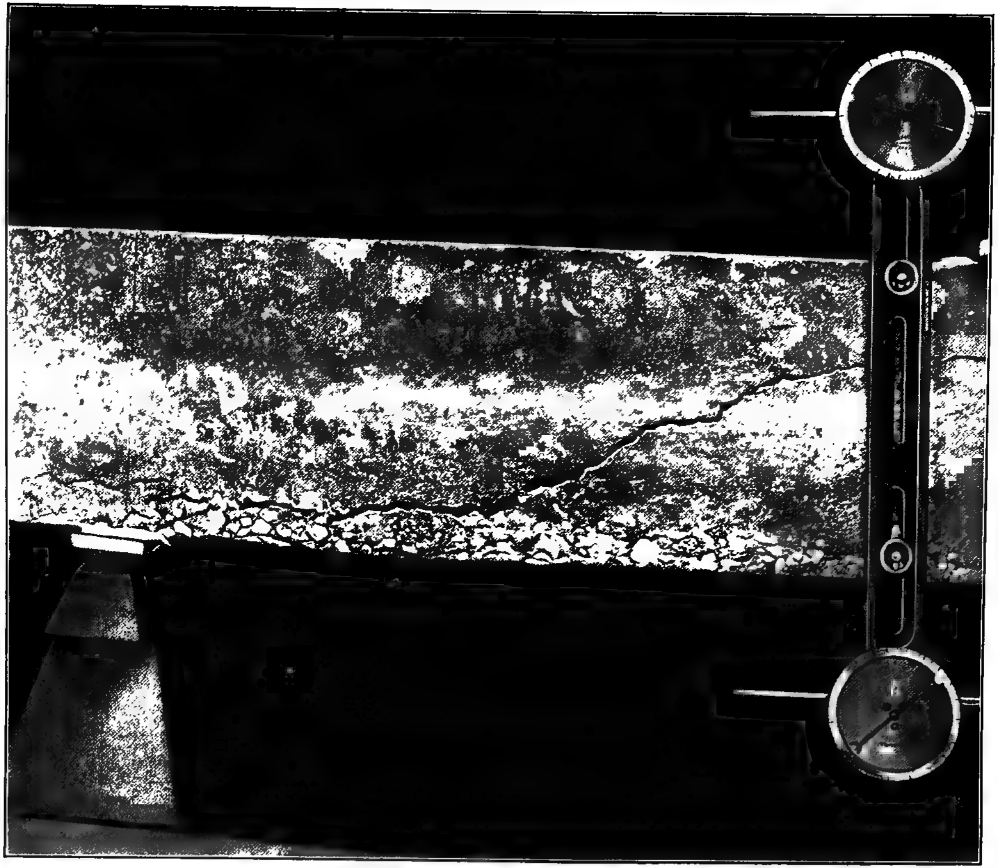


FIG. 20

ent on the presence of well-developed diagonal cracks, and is not a fundamental cause of failure.

The special stress conditions underlying this type of failure exist nearer the ends; hence, they would not affect the load-deformation diagrams, which for the usual test are based upon the deformations at the middle section of the span.

40. Primary and Ultimate Failure.—The proper classification of beam tests according to the manner of failure is an essential preliminary to the discussion of the results, and this classification must be based, not on the visible evidences of rupture, but on the critical stress conditions existing. To illustrate, in beams failing either by tension in the steel or by compression in the concrete, the most noticeable condition is the buckling of the concrete, as in Figs. 17 and 19. The first inference is that failure is due to crushing, but there is a great difference in the internal stress conditions of the beams, and this difference is the proper basis of classification. In one, the first variation from the conditions of the third stage of flexure is the development of the full compressive strength of the concrete; but in the other, the first variation is the development of the yield point of the steel, and the later spalling of the concrete is in this case the effect of necessary readjustment of stresses, as explained in the last part of Art. **37.**

Similarly, failure by diagonal tension may be accompanied or followed very closely by splitting in the plane of the rods. But this more apparent evidence of failure is dependent on the previous development of diagonal tension cracks, and hence is not the underlying, or primary, cause of failure.

41. Position of the Neutral Axis.—A very important item in the computation of the strength of reinforced-concrete beams is the position of the neutral axis. As mentioned before, the position of the neutral axis is not constant, but varies in position at different loads, owing to the fact that in concrete the stress is not exactly proportional to the strain. The particular value in which the designer is most interested is the one that locates the position of the neutral axis at or beyond the working load. It happens that the working load usually falls within the third stage of flexure. During this stage, the neutral axis is approximately constant in position in any one beam. This position is affected somewhat by the quality of the concrete, but depends primarily on the amount of reinforcement.

For rough computations, the neutral axis is frequently considered with 1 per cent. reinforcement to be below the compression face of the beam a distance equal to .45 of the effective depth of the beam when the safe load is applied.

It is generally assumed that in beams of considerable difference in depth, but having the same quality of concrete and the same amount of reinforcement, the neutral axis occupies the same relative position. This assumption has not been corroborated by a series of tests, but there is apparently no reason to doubt its correctness.

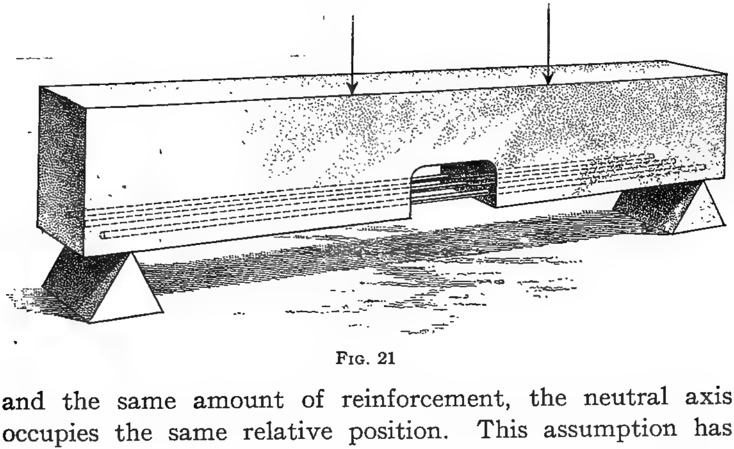


Fig. 21

42. Stresses in the Steel.—From the unit deformation in the steel, computed as described in Art. **31,** the average unit stress existing in the rods over the same gauge length may be calculated by multiplying the unit deformation by the modulus of elasticity. For example, a unit deformation of .001 inch per inch of length corresponds to a unit stress of 30,000 pounds per square inch for a modulus of elasticity of 30,000,000 pounds per square inch. To check the accuracy of the method of measuring and computing the unit deformations, two series of tests have been made. In these the reinforcing rods were exposed, as shown in Fig. 21, for a short distance within the gauge length of the usual apparatus for measuring beam deformations. Extensometers for measuring the deformation, or strain, were attached directly to the exposed rods. Simultaneous measurements were taken of the deformations by the two methods, namely, by the apparatus attached to the steel rods and by the yokes explained in Art. **29.**

TABLE IV

CHECK OF STRESSES IN STEEL

Beam Number	Load on Beam Pounds	Computed Stress Pounds	Stress in Steel, in Pounds per Square Inch, from Elongations in	
			Exposed Steel	Entire Gauge Length
49	8,000	25,900	23,000	27,000
53	8,000	26,100	25,200	29,400
62	8,000	26,100	25,800	23,100
Average	8,000	26,600	24,700	26,600

Table IV gives the unit stresses as obtained by these two methods. In the column headed Computed Stress is given the stress in the steel as computed from the actual load on the beam according to theoretical formulas. Considering the difficulty of obtaining exact agreement in tests involving the properties of concrete, the agreement between these three methods of computation is very close, and is an excellent check on the efficiency of the usual methods of measurement.

43. Yield Point of Steel.—In Table V are given the results of one series of tests that show the effect of the yield point of the reinforcing metal on the strength of reinforced-concrete beams. In this table are included only beams that failed by tension in the steel, because in beams failing in any other manner the steel is not the controlling factor.

In column 1 is given the beam number, which serves to identify it and distinguish it from other beams. Column 2 gives the percentage of steel present; column 3, the load that actually broke the beam; and column 4, the strength of the beam computed by a theoretical formula. It will be observed how closely the values in columns 3 and 4 correspond. Where there is any difference, the theoretical formula always gives values on the safe side. Column 5 gives the unit stress in the steel computed from the unit strain, or deformation, measured on the extensometer. Column 6 gives the yield point of the steel obtained by direct test. Since the unit stress in the steel, as given in column 5, is at the point where the beam was failing under maximum load, and the yield point of steel is about the same as the maximum load, the values in columns 5 and 6

TABLE V

RELATION BETWEEN YIELD POINT AND MAXIMUM STRESS IN STEEL

Beam Number	Area Metal Per Cent.	Maximum Load Pounds	Load for Ultimate Unit Stress Computed Pounds	Computed Unit Stress in Steel Pounds	Yield Point of Steel Pounds
1	2	3	4	5	6
21	.59	9,000	8,000	32,900	34,000
19	.59	9,200	9,200	37,400	34,000
7	.60	14,000	13,000	58,300	58,000
3	.60	14,000	12,000	52,600	58,000
16	.75	9,900	9,900	32,100	34,000
17	.75	10,000	9,500	29,300	34,000
20	1.00	20,900	20,000	65,200	58,000
2	1.00	20,600	19,000	58,800	58,000
15	1.20	18,400	15,500	35,000	35,000
10	1.20	16,600	14,500	32,000	35,000
5	1.20	15,000	13,000	30,600	34,000
14	1.60	17,200	17,000	30,200	34,000

should agree very closely. By referring to the table, it will be seen that this is a fact.

44. Compressive Strength of Concrete.—As has already been stated, in beams having a considerable percentage of reinforcement—say 2 per cent. or more—and sufficient provision for the diagonal stresses, failure occurs by the crushing, or spalling up, of the concrete at the top. In Table VI are given comparative values of the compressive strength developed in reinforced-concrete beams and compression cubes and cylinders, all of which contained concrete from the same mix. The beams were reinforced with 2½ per cent. of metal and developed compressive failures. The cubes were 4 inches on an edge, and the cylinders were 6 inches in diameter and 18 inches high. The tests were made at the University of Wisconsin and were discussed and reported by M. O. Withey. The stress in the reinforced-concrete beam was computed from the observed bending moment and deformations.

TABLE VI

COMPARATIVE VALUES OF COMPRESSIVE STRENGTH OF REINFORCED-CONCRETE BEAMS, ETC.

Beam Number 1	Maximum Load Pounds 2	Load for Unit Stress Computed Pounds 3	Computed Unit Stress in Beam Pounds 4	Compressive Unit Stress Average for Cube and Cylinder Pounds 5	Ratio of Unit Stresses in Beam to Pure Compression 6
1	7,800	7,000	1,740	1,285	1.35
2	7,000	6,000	1,440	1,325	1.09
3	8,000	7,000	1,780	1,360	1.31
4	8,760	7,000	1,830	1,715	1.07

Column 1 in the table gives the number of the beam, used for identification; column 2, the actual maximum load, in pounds, on the beam; column 3, the load it was calculated that the beam would carry, column 4, the stress it was computed that existed in the concrete from the extensometer readings; column 5, the unit compressive stress found by crushing cubes and cylinders of plain concrete; and column 6 the ratio of the values found in column 4 to the corresponding values found in column 5.

It is to be noted that in all cases a higher stress was developed in the beams than in the cubes, though the amount of the excess over the strength of the cubes and cylinders was somewhat variable. This same statement holds true not only for other tests in the series from which the illustrations were taken, but also for practically all other similar reliable tests in the United States. The probable reason for such an excess lies in the fact that in a beam only a small portion of the concrete is stressed to the maximum, while in a cube or a cylinder all the material is stressed to the maximum. Furthermore, even that small portion of the beam is stronger because of the support and restraint of the adjoining material. The four cases shown in Table VI illustrate well the variability of this excess, ranging in these cases from 7 to 35 per cent. of the strength of the cubes. Because of this lack of uniformity, it is not good practice to use a higher working stress in a beam than is used in pure compression. However, the fact that there is such an excess is of great value, because of the assurance that the ultimate strength of concrete as determined from comparatively small cubes may be safely used as a basis in the design of beams.

45. Shearing Strength of Concrete.—There are no well-established cases in which a concrete beam, either reinforced or unreinforced, has failed by pure shear in the concrete. During the first year or two of activity in the testing of reinforced concrete many failures of beams were reported as shear failures, but it is now well understood that these failures were, properly speaking, diagonal tension failures. The stress that actually reached its maximum was the diagonal stress, which is the resultant of the horizontal tensile stress and the vertical and horizontal shearing stresses. However, since this diagonal stress varies in much the same manner as the vertical shearing stress, it is convenient in design to provide for the diagonal stress by placing a limiting value on the shearing stress permissible.

In Turneaure and Maurer's "Principles of Reinforced-Concrete Construction" are listed a number of reliable tests from different sources in which failure has occurred by diagonal tension. From this list, 100 pounds per square inch may be taken as the average vertical unit-shearing stress at failure for beams having no diagonal reinforcement; for beams having reinforcement to resist the diagonal stresses, either in an inclined or a vertical position, this value varies from 130 to 340 pounds per square inch.

46. Splitting in Plane of Bars.—In earlier reports of tests, splitting of the beam in the plane of the reinforcement was not infrequently assigned as a cause of failure. As already mentioned, this splitting is not a primary cause of failure, but occurs only after the development of large diagonal cracks has brought about a readjustment of stresses. Without the diagonal crack, the splitting would not be possible. The primary cause of failure due to splitting is the development of the ultimate diagonal tensile stress.

47. Slipping of Bars.—In earlier tests failure was also frequently assigned to the development of the maximum-bond resistance. This report was based on the fact that where the concrete was broken from the ends of the beam at the completion of a test, it was found that the end of the rod had moved an appreciable distance. It is now believed that this motion follows after the maximum load has been passed and the diagonal cracks have developed to such an extent as to cause a decided readjustment of stress conditions. However, recent exact measurements of the motion of the end of the reinforcing rods relative to the concrete indicate a very small relative displacement. There are some evidences that this small relative displacement, which corresponds to a bond stress, has a very close relation to the possibility of development of a diagonal tension failure, but this theory is still in an uncertain state.

However, there have been reported cases in which the diagonal or vertical reinforcement has slipped owing to the actual development of the full-bond resistance. Such failures were caused by using plain bars and not taking the precaution to attach washers or to bend the ends so as to prevent such slipping.

It should be noted that the condition of the bond stress is much different in both the vertical and the diagonal reinforcement than it is in the main horizontal metal. In the former case, the length in which the stress in the rod is reduced from a maximum to zero is only a portion of the depth of the beam, while in the latter case, it is usually equal to one-third to one-half the span. In other words, the bond to be transferred per unit of length is usually much greater for the vertical reinforcement than for the horizontal reinforcement, although the maximum unit tensile stress in the metal may be much less.

TESTS ON SPECIAL BEAMS

48. Reinforced-Concrete T Beams.—Tests of **T** beams have been made in the United States by Prof. F. E. Turneaure at the University of Wisconsin and by Prof. A. N. Talbot at the University of Illinois, and in Germany by F. von Emperger. The indications of these tests are that with a flange width up to at least three times the width of the web, the ordinary theory applied to rectangular beams may be applied to **T** beams, the unit stress in the concrete at any level being considered uniform throughout the entire width of the flange. The majority of failures in these series of tests were by diagonal tension. This is in line with the theoretical conclusion that in **T** beams special attention must be given to the design of the diagonal reinforcement if the full benefit of the tension in the steel and of the compression in the concrete is to be obtained.

49. Continuous Beams.—So few tests of continuous reinforced-concrete beams have been made that designers have had to depend on theoretical considerations of continuity as applied to beams and girders of homogeneous material. Only one such series of tests has been made, namely, at the University of Wisconsin, in 1907. These beams were tested on an 8-foot span with an overhang of 3 feet at each end. The loads were applied at the one-third points of the span and at the ends of the overhangs. The percentage of reinforcement was large, being almost 2 per cent., the intention being to develop the full compressive strength of the concrete. The diagonal reinforcement varied from none or the use of light expanded metal to the use of heavy vertical stirrups and the bending up of a portion of the main rods in the same beam. The majority of failures were by diagonal tension, none of the beams failing by compression in the concrete. The one important lesson to be learned from this series of tests is the necessity of sufficient provision for the diagonal stresses.

PLAIN-CONCRETE COLUMNS

50. Methods of Testing. — In actual construction, columns are nearly always reinforced with steel, and on that account comparatively few tests have been made on plain-concrete columns. The methods of testing are identical with those for reinforced-concrete columns, which will be described presently. The observations made for plain-concrete columns are the maximum load and the compressive deformations at various loads.

51. Results. — Plain-concrete columns fail suddenly without previous warning. The two typical manners of failure are illustrated in Figs. 22 and 23. That shown in Fig. 22 has been referred to as a diagonal shearing failure, and that in Fig. 23 as a direct-compression failure. The fact that unreinforced columns usually fail very suddenly indicates a necessity for low working stresses in such members.

52. Fig. 24 shows stress-deformation curves for two plain-concrete columns tested at the University of Illinois. In these cases, the load was increased continuously from zero to the maximum without release of the load. The height from the base line to the curve represents the unit stress produced, and the horizontal distance from the left-hand border line to the curve represents the unit strain, or deformation, produced. It may be stated that the deformation measure-

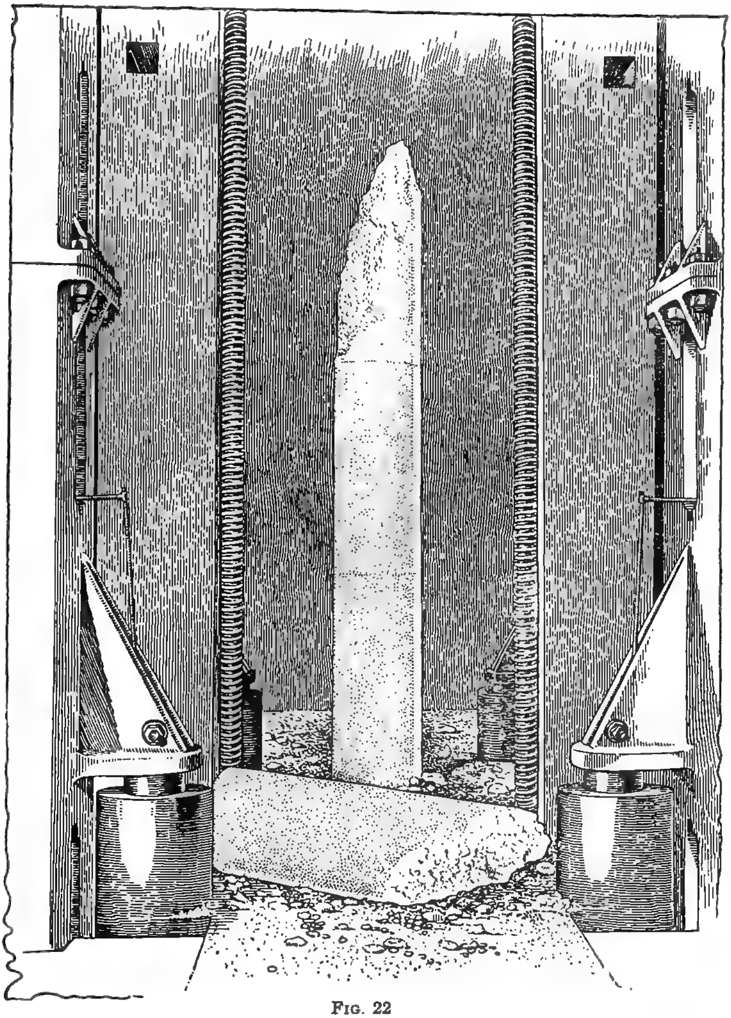


Fig. 22

Fig. 23

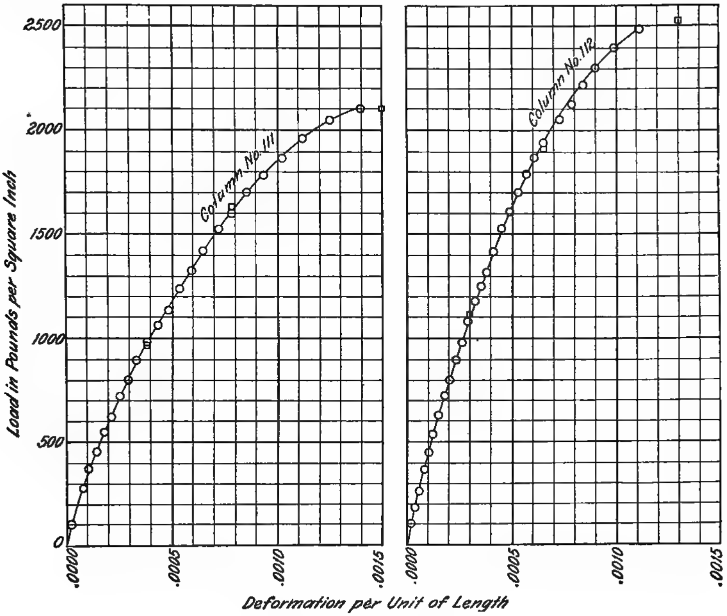


FIG. 24

ments indicate that the action of plain columns is similar to that of compression cylinders under corresponding conditions.

53. Fig. 25 illustrates the effect of variation in the quantity of cement on the ultimate strength of the column. In this diagram, the figures express the proportion of the material by volume, and the horizontal scale is the ratio of the weight of cement in terms of the combined weight of the sand and stone. The

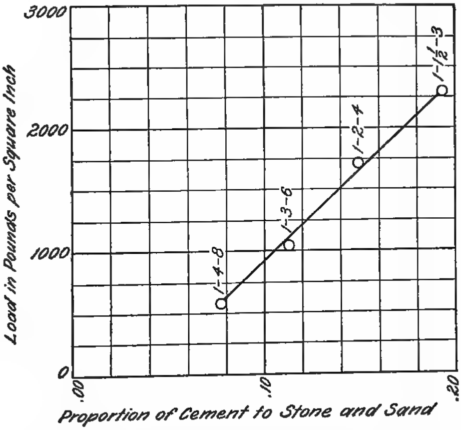


FIG. 25

vertical scale is expressed in ultimate load per square inch. It should not be inferred from this single series of tests that columns made from all aggregates or from all brands of cement will follow the same relation, but the conclusion to be drawn is that increasing the quantity of cement produces a uniform increase in the strength of the mixture.

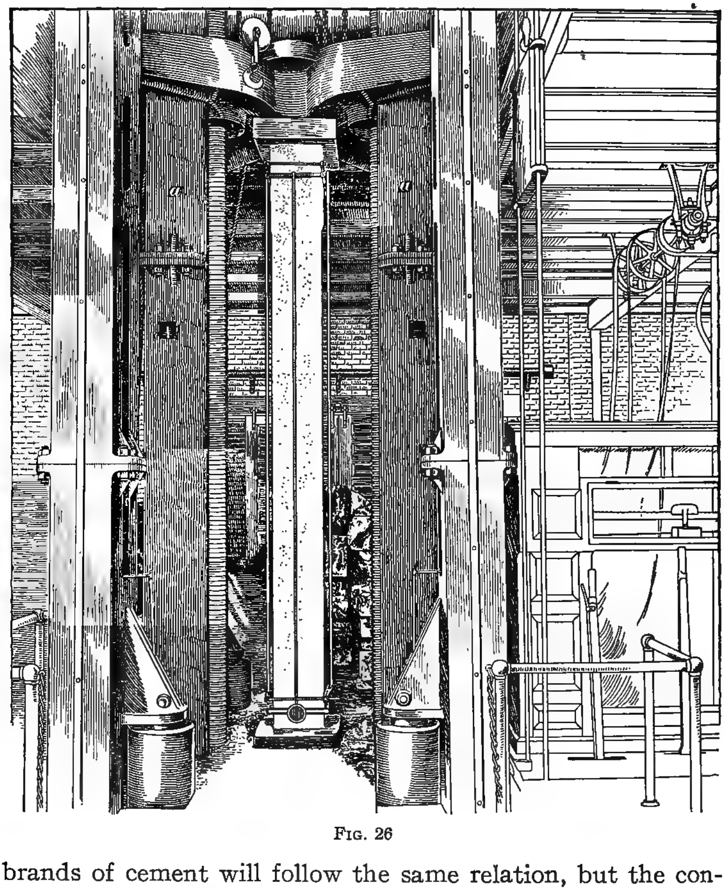


Fig. 26

REINFORCED-CONCRETE COLUMNS

METHODS OF TESTING

54. Position of Column Under Test.—At the Watertown Arsenal, reinforced-concrete columns are tested in a horizontal position, under which condition it is necessary to counterbalance the weight of the column at different points along the length to prevent bending as in a beam. At the University of Illinois, the columns are tested in a vertical position, as shown in Fig. 26. In principle, the machine used in making this test is the same as that described for testing beams, but the pulling head is braced against four columns *a*, one on each corner, to prevent it from twisting. The weighing platform is level with the floor, and the operating mechanism is located below the floor and to the right of the upright frame.

55. Measurement of Deformations.—In Fig. 27 is shown one type of apparatus for measuring the compressive deformations in columns. It consists of two yokes fastened at the upper and lower ends of the column, respectively. A light bar is fastened rigidly to one yoke at one end, and has at the other end a flat steel strip that passes over a small cylindrical roller that rotates in bearings attached to the second yoke. Attached to the roller is a pointer that moves about a dial, which is provided with means of reading it accurately. Usually, two measuring devices are used. These are arranged in such a way that the deformation is measured

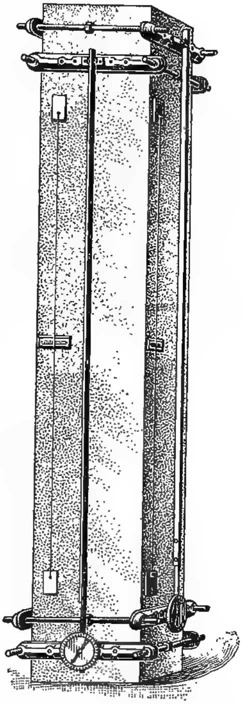


Fig. 27

either at the middle of each of the four faces of a square column or at the opposite ends of two mutually perpendicular diameters of a circular column. The lateral deflection of the column is also measured by a string in front of a graduated mirror, as shown in Fig. 27.

INTERPRETATION OF RESULTS

56. Uniformity of Results.—The results of column tests have shown great variation, and no one series of experiments should be considered sufficient either to establish or to disprove a principle of design. Several corroborative series are needed before the conclusions should be accepted as final, although the results of a series of well-managed tests may well be taken as a tentative basis of design pending the obtaining of more evidence.

The following extracts from Bulletin No. 10 of the University of Illinois Engineering Experiment Station will serve to illustrate the variation that may occur even in carefully conducted tests:

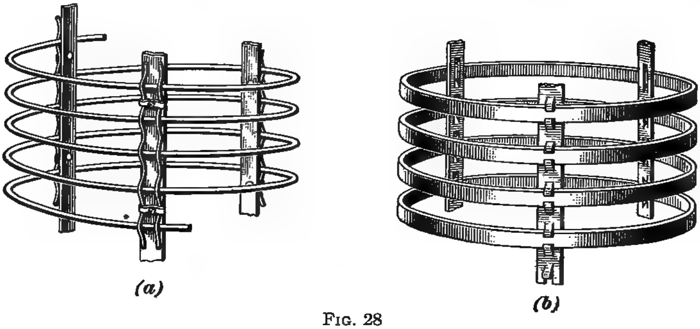

(a)
(b)

FIG. 28

"The average maximum load for the plain-concrete columns tested was 1,553 pounds per square inch. The lowest value is 30 per cent. less than this average, and the highest 39 per cent. more. The average variation from the average strength is 18 per cent.

"The average stress taken by the concrete alone is for the reinforced columns 1,290 pounds per square inch. The lowest value is 26 per cent. below the average, the highest is 30 per cent. above, and the average variation from the average is 16 per cent."

57. Classes of Reinforced Columns.—In analyzing the results of column tests, it is necessary to distinguish

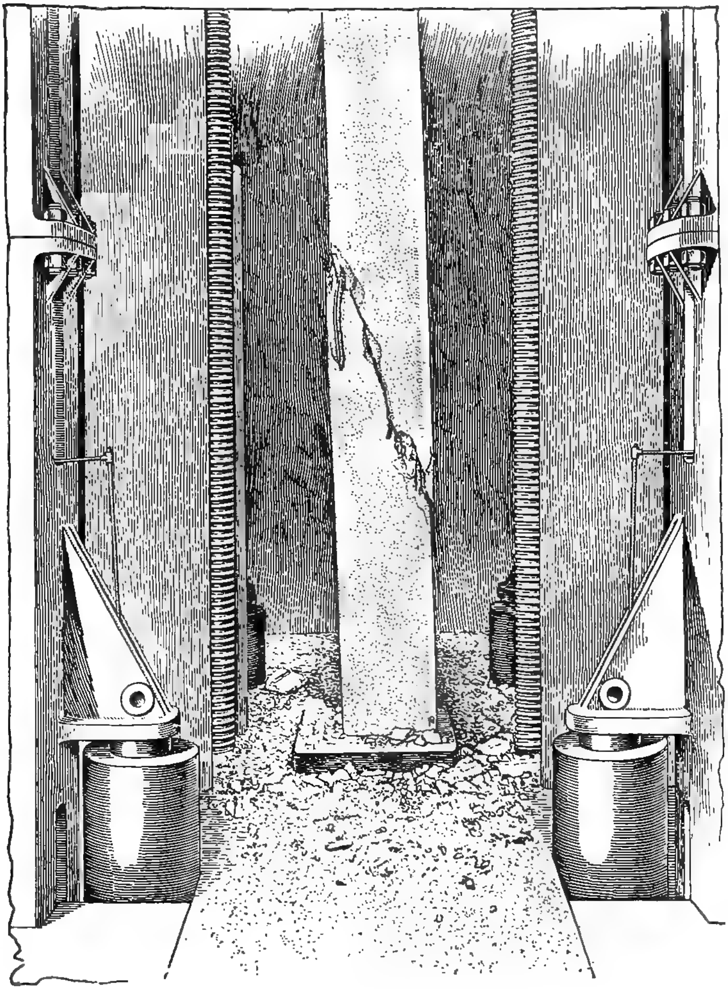


FIG. 29

between several types of reinforcement. By *longitudinal reinforcement* is meant the use of steel rods placed parallel to the axis of the column. By *hooped reinforcement* is meant

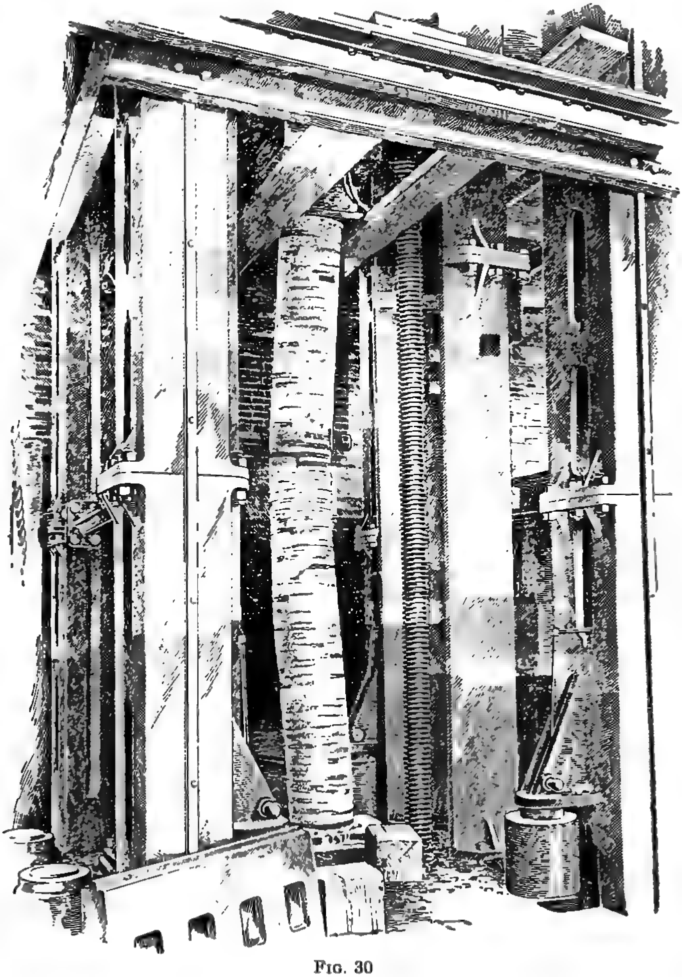


Fig. 30

the use of circular bands of metal placed in planes approximately at right angles to the axis of the column and near the surface. In Fig. 28 are shown two types of hooped reinforcement, which, for convenience, may be distinguished as *spiral hooped*, as in (*a*), and *band hooped*, as in (*b*). Sometimes a combination of longitudinal and hooped reinforcement is used in the same column.

58. General Characteristics of Failure.—In Fig. 29 is shown a view taken after the development of the maximum load for a column reinforced with vertical rods only. Failure is usually first indicated for this type of column by vertical hair cracks, after which the concrete commences to bulge at the section of failure. The final characteristic consists of increased bulging, to such an extent that the rods buckle or the column shears across a diagonal plane, as here indicated.

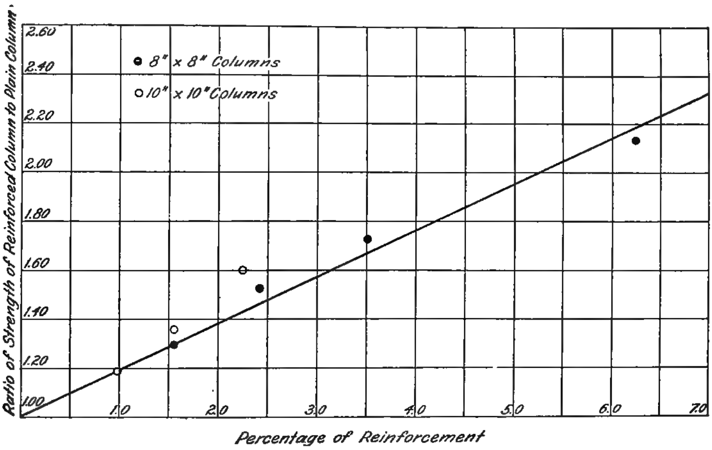


FIG. 31

Fig. 30 shows a column with hooped reinforcement after it has been loaded beyond its ultimate strength. The first visible evidence of failure during the tests is the scaling off of the concrete that covers the spirals. This occurs at a load somewhat above the ultimate strength of a similar unreinforced column. The lateral deflection of the column from a straight line is apparent before the maximum load is reached—in many cases considerably before.

59. Strength of Columns Reinforced With Vertical Rods.—The diagram shown in Fig. 31 gives the results of a series of column tests made at the Massachusetts Institute of Technology, and as arranged by Turneaure and Maurer in their work entitled, "The Principles of Reinforced Concrete." The columns were square, either 8 in. × 8 in., or 10 in. × 10 in., and made of 1–3–6 concrete. The rods were either plain square rods having an ultimate strength of 60,000 pounds per square inch, or twisted square rods having an ultimate strength of 80,000 pounds per square inch. The number of rods in each column was either one placed at the center or four placed at the corners of a square one-half as large as the cross-section of the column. The age at the time of testing was approximately 30 days.

The horizontal scale represents the percentage of reinforcement, based on the ratio of the cross-section of the rods to the gross cross-section of the column; and the vertical scale gives the ratios of the ultimate strength of the reinforced columns to the probable strength of a plain-concrete column of like material.

This series of tests indicates that for the type of column under consideration the increase of strength above that of a plain-concrete column varies directly with the percentage of reinforcement. The rate of this increase of strength will depend on the relative moduli of elasticity of the steel and the concrete in compression, but the average increase is indicated by the diagonal line. No other series of tests with a range of percentage of reinforcement sufficient to serve as a check on this conclusion has been made so far in the United States. The conclusion is in conformity with the assumption made by reliable designers, both before and since the preceding series was made public.

Tests made at the University of Illinois, in 1906, do not indicate as great an effect due to increase of amount of reinforcement as in the preceding series at the Massachusetts Institute of Technology. However, in this series, the range of percentage of reinforcement was small, which may account for this difference.

60. Strength of Hooped Concrete Columns.—Considère, in France, was the first person to publish tests showing the great increase in ultimate strength due to the use of spiral reinforcement. These tests have been given wide publicity and have received general acceptance. Considère's most important conclusion is that steel in the form of spiral reinforcement is 2.4 times as effective as steel in the form of longitudinal reinforcement. Professor Talbot reports concerning his own tests of 1907 that, "so far as ultimate strength is concerned, hooping adds two to three times as much strength to the column as does an equal amount of longitudinal reinforcement."

TABLE VII

STRENGTH OF HOOPED CONCRETE COLUMNS

Description of Column	Ultimate Unit Strength Pounds per Square Inch
Plain concrete	1,413
Reinforced with 13 hoops	2,232
Reinforced with 25 hoops	3,428
Reinforced with 47 hoops	5,289

Table VII, which shows results of tests made at the government laboratory at the Watertown Arsenal, will serve very well to illustrate the marked increase in strength for this type of reinforcement. The concrete used was a 1–2–4 mixture and was 6 months old. The columns were circular, 10½ inches in diameter, and 8 feet long. The metal in thirteen hoops was approximately equal to 1 per cent. The value given is the ultimate strength in pounds per square inch.

61. Deformation of Reinforced-Concrete Columns. In Fig. 32 are shown the stress-deformation curves for two columns reinforced with approximately 1 per cent. of metal, all in longitudinal position. In general, it may be stated that the characteristic curve for columns so reinforced differs from the characteristic curve for unreinforced columns in that the load corresponding to a given deformation is greater in the reinforced column than in the unreinforced column. This is as would be expected. The cross-section of the metal reinforcement and the unit deformation for a given load being known, the proportion of the load carried by the steel may be determined by multiplying the unit deformation by the modulus of elasticity. Assuming the remainder of the load to be uniformly distributed over the concrete, the unit stress developed therein may be determined. The curve showing the relation between these computed unit stresses in the concrete and the unit deformation of the column corresponds very closely with the usual stress-deformation relation of plain concrete in compression. It will be noted, however, that the vertical scale of the diagrams shown in Fig. 32 is not the unit stress in the concrete alone, but the load per square inch of cross-section, which might be called the unit stress developed by the two materials working together.

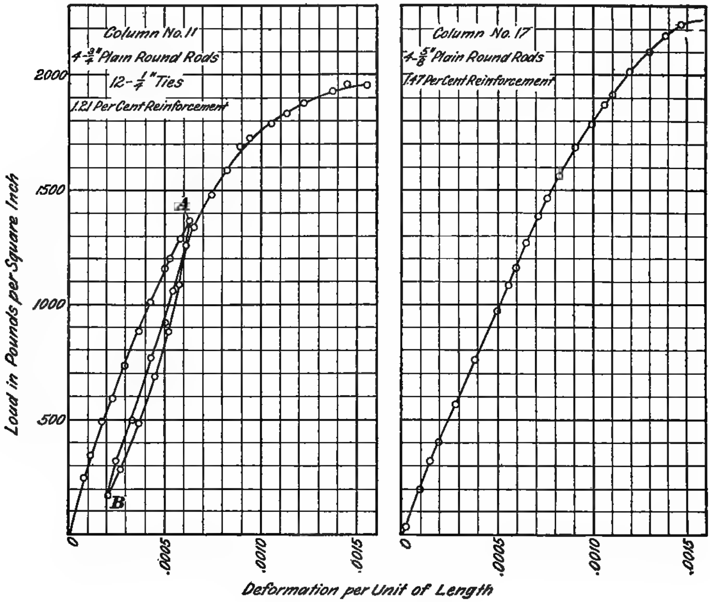


FIG. 32

In the curve that represents the column designated as No. 11, that is, the curve at the left-hand side of the figure, it will be noted that after the column was loaded in the testing machine to the point *A*, or, in other words, when the load per square inch was about 1,375 pounds, the testing machine was reversed and the load was reduced. The load was finally reduced to the point *B* on the curve, when it equaled about 170 pounds. It can be seen from the direction of the curve between *A* and *B* that if the load had been further reduced to zero, the deformation per unit of length would not have been zero; that is, the column would have had a permanent set. From the point *B* the load was again increased until the column failed. The circles shown along the curve indicate at what points readings were taken.

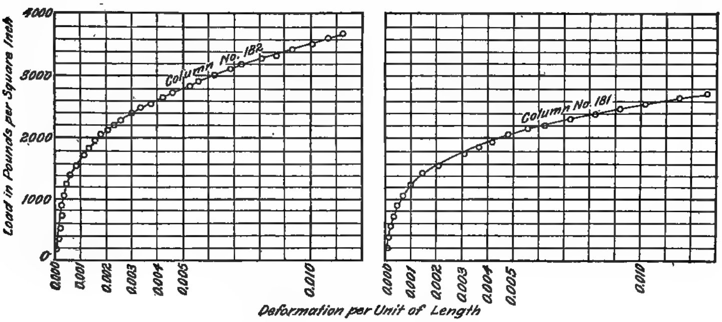


FIG. 33

62. Fig. 33 shows stress-deformation relations for two hooped columns having approximately 3 per cent. of metal. Concerning these characteristic curves, as compared with those for plain concrete, it may be said that up to the unit deformation corresponding to the ultimate strength of a plain-concrete column, the curves are very similar. For unit stresses greater than this, the curve is approximately straight. Of this feature Professor Talbot says: "In hooped columns, the hooping does not come into action to any great extent before a load equivalent to the ultimate strength of plain concrete, or a little below this value, is reached. Beyond this load, the column shortens rapidly and the hooping receives increased stress, and its function in restraining the concrete laterally becomes important. If the column maintains a straight position, the limit of strength may be expected to come when the stress in the hooping exceeds the yield point of the metal. If the column deflects laterally through eccentric loading or lack of uniformity in the concrete or in the distribution of the hooping, a lower strength will be found."

63. Effect of Length on Strength of Reinforced-Concrete Columns.—Only one series of tests has been made to find out what effect the length of a column reinforced with longitudinal rods has on its strength. The tests were made at the Massachusetts Institute of Technology, and lengths were used up to twenty-five times the diameter of the column. There appeared to be no noticeable decrease in strength for the greater lengths. This conclusion has been accepted as a fact by many good designers but it should be checked by more extensive tests.

Tests of hooped concrete columns made at the University of Illinois indicate that there is a noticeable decrease in ultimate strength of such columns as the length is increased. This decrease was noticed even for comparatively short lengths.

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NOTE.—In this volume, each Section is complete in itself and has a number. This number is printed at the top of every page of the Section in the headline opposite the page number, and to distinguish the Section number from the page number, the Section number is preceded by a section mark (§). In order to find a reference, glance along the inside edges of the headlines until the desired Section number is found, then along the page numbers of that Section until the desired page is found. Thus, to find the reference "Acute angle, §5, p3," turn to the Section marked §5, then to page 3 of that Section.

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